MODERN·BUILDINGS·THEIR PLANNING·CONSTRUCTION·& EQUIPMENT

EDITED·BY·G·A·T·MIDDLETON·A·R·I·B·A

MODERN BUILDINGS

THEIR PLANNING, CONSTRUCTION AND EQUIPMENT

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BY

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ETC. ETC.

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PROFUSELY ILLUSTRATED

VOL. IV

PART I. PUBLIC BUILDINGS
PART II. STEEL CONSTRUCTION
PART III. FIRE-RESISTING CONSTRUCTION

LONDON:
THE CAXTON PUBLISHING COMPANY
CLUN HOUSE, SURREY STREET, W.C.

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MODERN BUILDINGS

VOLUME IV

PART I

PUBLIC BUILDINGS

CHAPTER 1

PUBLIC LIBRARIES

There have been in former times periods of activity in ecclesiastical building, in military building, and in domestic building, but never until the present has there been in England any really considerable amount of building for public purposes. In this respect England differs from some continental countries, particularly Belgium, where the greater public buildings were erected in the sixteenth century. As one small result of this no book which pretends to deal with modern buildings can possibly neglect these great public works, and it is proposed to consider them here in a series of chapters commencing with buildings of a more simple, and proceeding to those of a more elaborate character.

Possibly the most simple of the public buildings of the day are the Public Libraries ; and they are also the most general. They are recognised as being a social necessity in every community, large or small ; while the tendency, of the moment at any rate, is towards the multiplication of small libraries rather than to the erection of large ones, thus bringing books to the readers instead of taking the readers to the books. In a paper read by Mr. H. T. Hare, F.R.I.B.A., before the Librarians' Association in August 1905, he enumerated the leading requirements as follows :—

I. Ample space or area in all parts.
II. Abundance of light, air, and ventilation.
III. Facility for supervision and working.

With regard to the first point, which he said sounded like a truism, he contended that it was a desideratum specially to be insisted upon. He said that almost all public libraries were much too closely packed for the comfort, health, and free circulation of readers ; and the second requirement is similar to it. In the matter of supervision, there has been in the past a good deal of misconception. At one time it was a general idea that the superintendent should be so placed as to be able to act as a supervisor of all that went on from his office ; but such an arrangement is exceedingly uncomfortable, and by no means calculated to facilitate his work, while in large and simply planned rooms the public will to a great extent supervise themselves. Beyond this the *possibility* of the controlling eye of the staff being on them is in most cases sufficient.

All libraries of moderate size include a lending library, or loan department, with ample space for bookstands approached only by the staff, who are separated from the public by a counter, along the outer edge of which is ranged a series of indicators. These consist of tiny shelves or pigeon-holes, into each of which fits a small tray, numbered both on back and front with the same number, each number representing one of the books in the library. When a book is borrowed, the borrower hands in a small card to the attendant which exactly fits into one of the trays. The tray corresponding to the book he requires is taken out of its pigeon-hole, the card is put into it, and the tray returned. The two ends of the tray are painted in different colours, so that if, say, the tray is empty and the book is on the library shelves, the blue end of the

tray faces the public and the red end of the tray faces the attendant. When the book is borrowed and the tray is returned with the borrower's card in it, it is replaced so that the red end faces the public and the blue end faces the attendant. Thus by consulting the indicators both borrower and attendant know at once whether any particular book is on the shelf or has been lent. When the borrower returns the book and receives his card in exchange the little tray is again reversed.

The amount of space necessary in a Lending Library varies considerably, according to its arrangement. It is best for the books to be upon stands which are erected in the middle of the room and top-lighted, the "staff entrance," while a counter-flap gives access from the hall.

Another very important room is the reference-room. This is the only one which needs isolating, and putting, if it be possible, in a quiet spot, as it is used by persons desirous of engaging in serious study. At Nelson it is immediately accessible from the librarian's room, by which it is overlooked, as well as from the public hall, and this is exceedingly convenient; but in many libraries it is found well for it to contain shelving for a large number of important reference books which are not to be found in the lending library, they being of too great value to issue indiscriminately. It is important that in

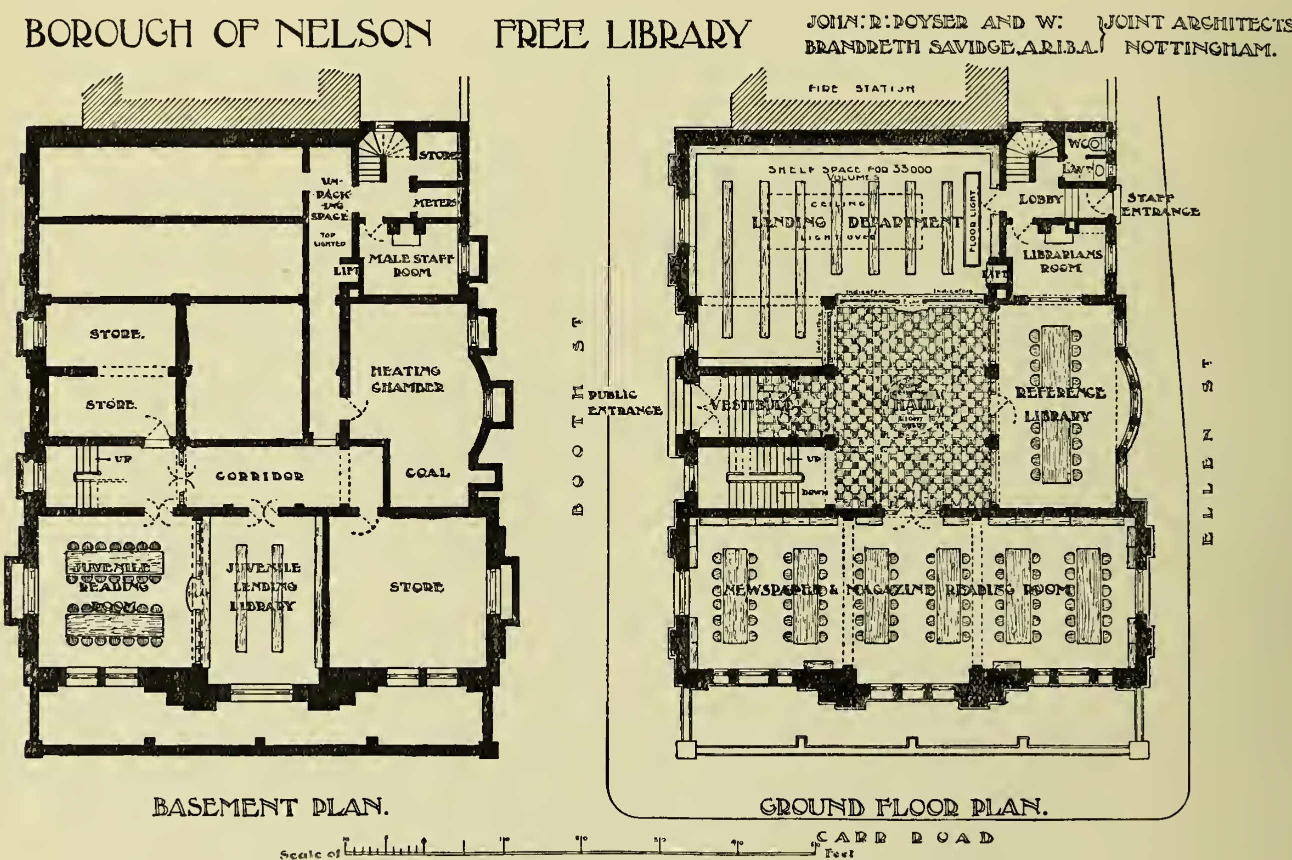


FIG. 1.

stands being accessible on both sides, as is shown in Fig. 1, which illustrates the basement and ground floor plans of the new library at Nelson, designed by Messrs. Poyser & Savidge. Each of the stands has shelves for two ranges of books, so that books are found to right and left of each gangway. The shelves are generally graded to take books of the usual smaller sizes, while any shelves round the walls are arranged for larger volumes, folios as a rule occupying the bottom shelves. The arrangement shown at Nelson may be taken as typical of that which is now usual, the public entering directly into a large hall, along one side or end ot which is the borrower's counter. The lending department can be reached by the staff through a door from a the reference-room there should be ample table space for each student, and many would prefer that separate tables were provided, which would necessitate a floor space of something like 20 square feet each. With tables such as those shown at Nelson, of considerable width, and seating students on each side, a floor space of 15 feet per student will suffice. Possibly the best of all arrangements is that used in the British Museum reading-room, with shelves and book rests in front of the students, provided with all necessaries; but they are expensive and are rarely to be found in small libraries, where open tables are generally sufficient. These and other library fittings will be more fully dealt with in Volume V., under the heading of "Equipment."

It is a common thing to give up a whole section of a library to newspapers and magazines, the papers being arranged upon stands and the magazines and other periodicals upon tables, the stands being some of them in the middle of the room and some round the walls. Many librarians, however, contend that the slopes for the popular papers should be placed round the wall only, as they occur in the Nelson Library (Fig. 1), so as not to interfere with a general comprehensive view by the staff of the whole place in case of necessity, and to secure that supervision of the public by themselves which has already been referred to. Each large daily paper requires 4 feet run of slope, while the magazine tables have to be of a minimum breadth of 3 feet, if readers are to sit on both sides, while a minimum length of 2 feet must be allowed for each chair. The space between the tables should be at least 7 feet, and between the edge of the tables and end of newspaper slopes at least 6 feet.

It is often convenient to separate the magazine-room from that given up to the newspapers and more popular periodicals by a glazed screen, so as to provide greater quietude, as there are always a considerable number of persons passing in and out of the newspaper-room. In fact, there is a good deal to be said for the employment of these glazed screens instead of solid partitions, provided they can be made noise proof, as they allow of diffusion of light and easy control.

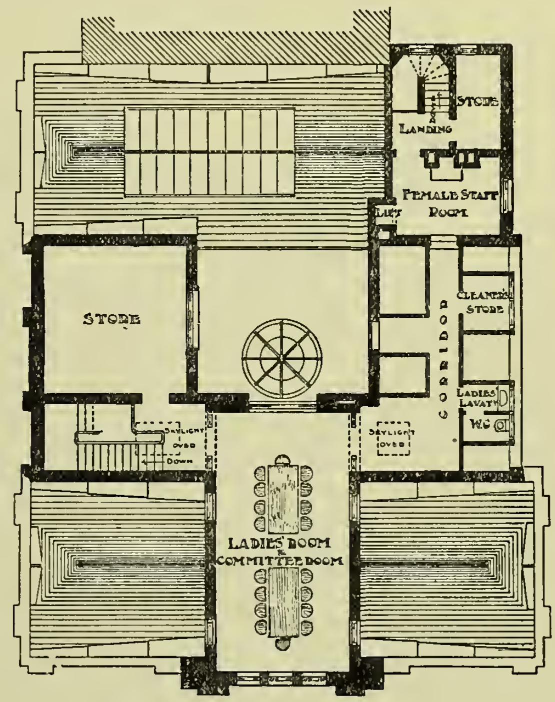
A careful examination of the plan of the Nelson Library will show how exceedingly simply and logically everything is arranged round the central hall, how easily the public can enter any desired room, and how, by slightly recessing the lending counter, the crowd of borrowers is kept from interfering with persons who are passing to and from the other departments. There is also a certain formality in the arrangement, which is axial in both directions, ensuring simplicity, and giving opportunity for true architectural treatment, both externally and internally, which was well taken advantage of, as may be seen by reference to the elevations (Fig. 3).

It will be noticed, too, that a staircase for public use opens out of the large hall and goes down to a basement, where a juvenile reading-room and lending library are located. This is an exceedingly good arrangement, as it keeps the noisy children away from the more serious workers, while providing for them a certain amount of supervision from the attendant in charge of their lending library. There is no greater nuisance in the ordinary small library than that of having a number of children clamouring for books from the lending department, and passing in and out of the reading-rooms. The objection to giving them a special room, such as this, is the constant expense involved in providing an attendant to look after them.

The basement is also utilised for large storerooms, which are always essential for receiving and binding books and filing newspapers and magazines until such time as they are ready for binding. A well-lighted heating chamber is also placed in an accessible spot in the middle of the basement, which can be reached by a small staircase from the staff entrance as well as from the main staircase, while a lift leads from an unpacking space in a broad passage up to the lending department. At the foot of this back staircase a small male staff-room is provided, and above it on the ground floor is a librarian's room. The staff entrance at this point is very cleverly managed from a back street. Messrs. Poyser & Savidge themselves remark that the problem which had to be solved was

BOROUGH OF NELSON: FREE LIBRARY:

JOHN: B: POYSER AND W: BRANDRETH SAVIDGE, A.R.I.B.A. } JOINT ARCHITECTS, NOTTINGHAM



FIRST FLOOR PLAN

Scale of 0 5 10 20 30 40 50 60 Feet.

Fig. 2.

to provide a considerable amount of accommodation on an exceedingly limited site, and as far as possible on one floor, there being streets on three sides of the site and a public building on the fourth. They say that they do not consider that the plan is by any means an ideal one from the point of view of supervision, but the narrowness of the site necessitated that either this should be sacrificed or that the space available for public rooms should be curtailed to an undesirable extent. The question of supervision has, however, frequently been made too much of, and in this instance the librarian has easy access to all parts, if necessary, without appearing to act as a policeman, while the

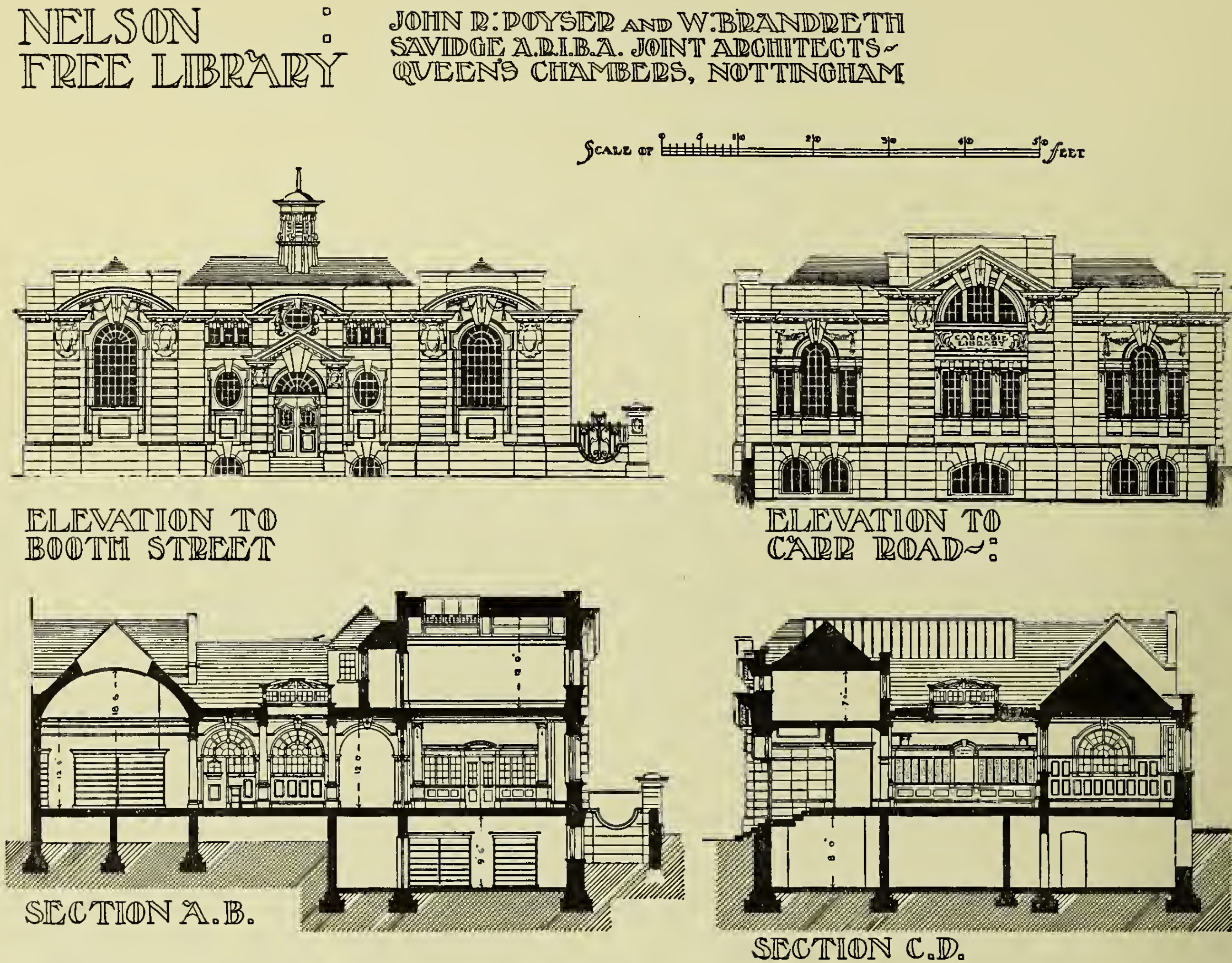


Fig. 3.

attendants in the lending department have oversight of the central hall and of the way down to the juvenile department.

On the first floor (see Fig. 2) a good sized quiet room, reached by the main staircase, is reserved for ladies and for use as a committee-room, while a large storeroom is provided over the entrance. A communicating corridor gives access from this room to the female staff-room, which is above the librarian's rooms. All this accommodation is practically obtained within the roof, and the plan is shown principally in order to illustrate the method of lighting the hall and the lending department, which can also be seen on the section given in Fig. 3.

The ground floor is 12 feet high, except in the lending department, which has a segmental ceiling, with its springing 12 feet 3 inches above

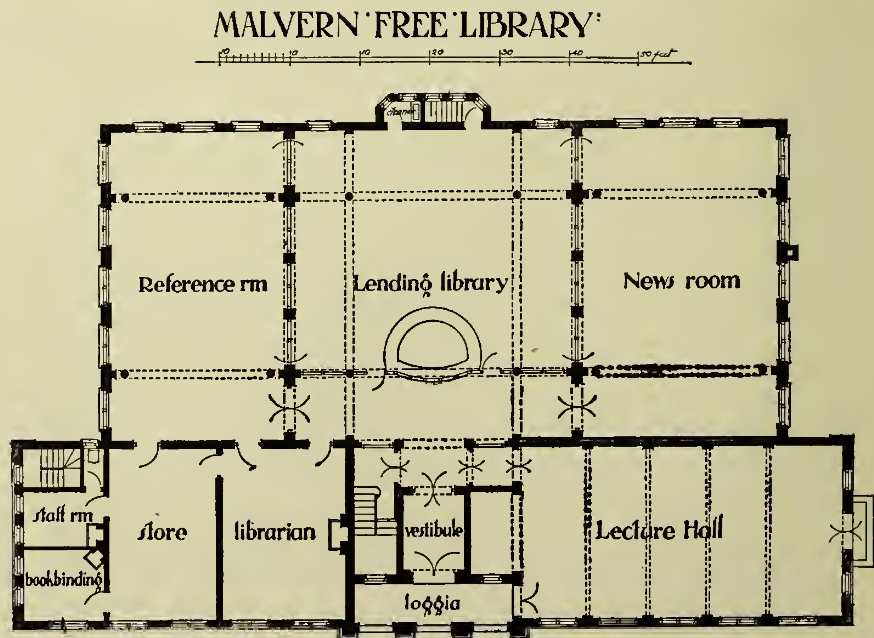


Fig. 4. *H. A. Crouch, Architect.*

GROUND PLAN

the floor and a total height of 18 feet 6 inches. The basement is figured as 9 feet 6 inches high at the juvenile lending-library, while on the first floor the ladies' reading-room has a total height of 12 feet, though the storeroom is only 7 feet high. These heights may be accepted as the ordinary standards for libraries of about this size. Where a one-storey building is impracticable, Mr. Hare says that the lending library is usually placed on the ground floor, and the reference library, librarian's room, bookstore, etc., on the first floor; but he sees no reason why it should be regarded as *essential* that the lending library should be on the ground floor, and believes that there ought to be more space provided than is usual. He consequently suggests that the whole of the ground floor should be given up to a large reading-room for all classes, except reference.

Another important thing in all libraries is to provide sufficient light. Where this can only be obtained from side windows no room should be more than 30 feet in width, and if the street is narrow and high buildings obstruct the view, 20 feet is quite enough. In each case a style of architecture should be adopted which enables the windows to be glazed with large sheets of clear glass. The window-sills should be about 6 feet from the floor, especially in newsrooms, to allow the provision of wall-reading stands, and to prevent idlers from standing about looking out at the windows.

Another simply planned small library is that at Malvern, designed by Mr. H. A. Crouch (see Fig. 4). The lending library in this case is placed exactly opposite the entrance, and there is perhaps some risk that borrowers, when consulting the indicators, would find themselves in the way of others of the public who wished to enter either the newsroom or reference-room, which occur to right and left of it. The librarian's room is accessible from the reference-room, but the staffroom is only to be reached through a store. In this library a lecture hall has been added, with a

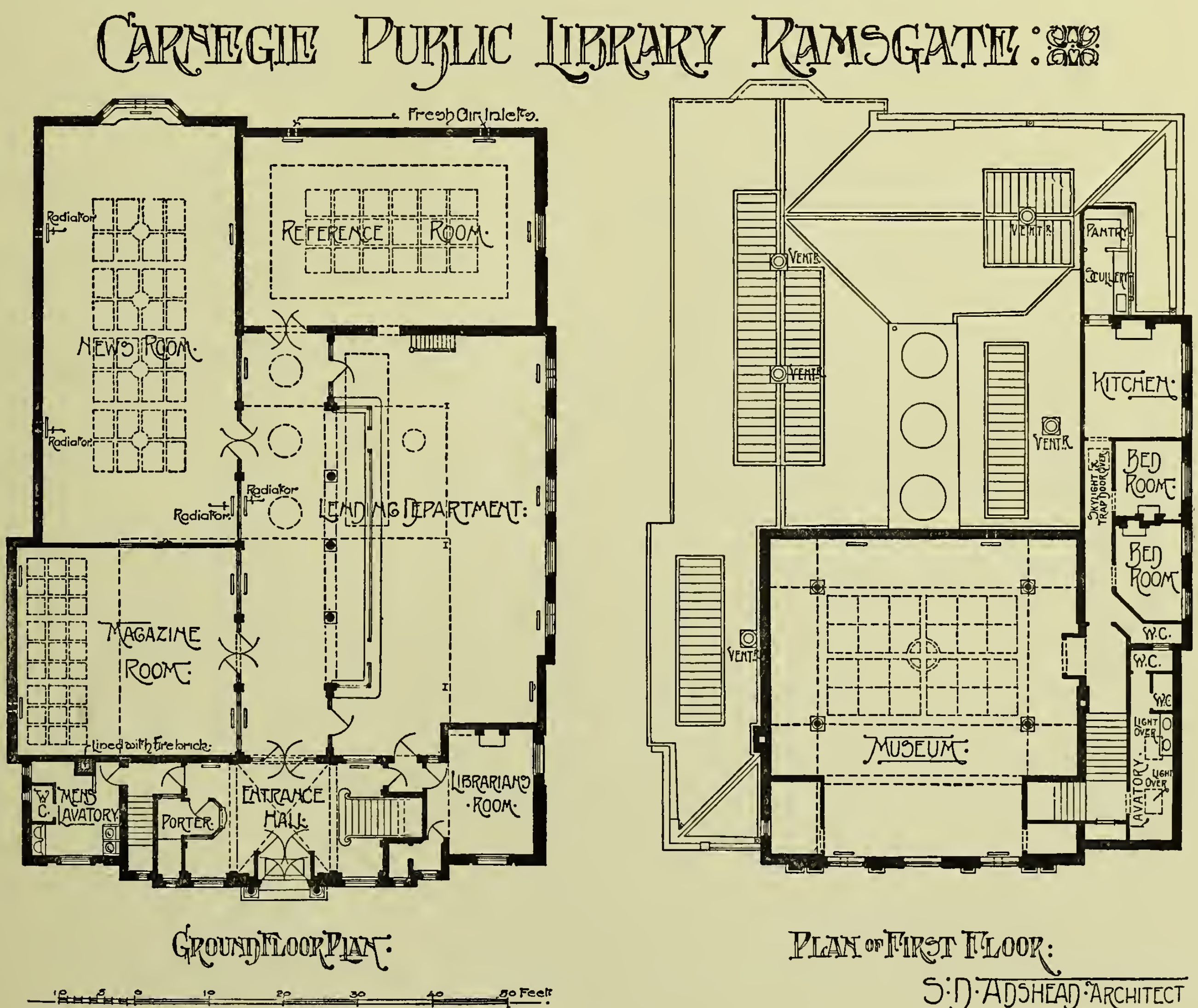


Fig. 5.

platform and three entrances. This is not often done, as to some extent it is foreign to the purposes of the Libraries Act, so that it is difficult to justify the expense of its provision and upkeep from the rates. If it is demanded, however, it could not be much better arranged than as at Malvern, the stage end being accessible from the main library entrance and from the body of the building, while the other end can be reached by the general public externally. It is separated from the newsroom by a hollow wall, in order that sound may not penetrate.

Fig. 5 illustrates a somewhat larger library at Ramsgate, designed by Mr. Stanley D. Adshead, for a site which was somewhat difficult to light. The plan is adopted here of cutting off the entrance hall from the library proper by a screen, placing the librarian's room so that it can be reached from the inner hall and from the lending library only. A staircase rises out of the hall on one side, screening the librarian's room; and by means of this access is obtained to a museum, and to a small residence for the caretaker on the first floor. On the opposite side of the hall is a small porter's or ticket office, while the librarian's room is balanced by a large and well-lighted lavatory. From the hall, opposite the entrance, a long wide corridor or inner hall is entered, and the remaining rooms are planned off this much as are classrooms in a school built on the corridor system. The large lending department is on one side, with a recessed counter, so that borrowers would not interfere with the corridor space when consulting the indicators, which are top lighted. On the other side are the magazine and newsrooms, with good top lighting, while at the extreme end of the corridor is the reference-room, also top lighted, in the quietest possible spot. This has a gallery round it, which is reached by a small staircase from the lending department, and serves for the additional storage of books, of which a considerable number are placed in shelves round the room and accessible to readers. Top lighting, such as is here resorted to largely, is on the whole perhaps the most satisfactory of any, and it will be noticed that there is no crowding of the tables, ample space being provided for readers in all departments.

Fig. 6 illustrates the small Castle Moor Library, on a corner site, designed by Mr. Jesse Horsfall, F.R.I.B.A. The site here is cramped, and as a result side lighting has to take the place of top lighting, except to a small extent in the lending library. This has controlled the arrangement of the tables in the reading and newsroom, while it has not been possible to make special arrangements either for a staff entrance, for a private staffroom, or even for a librarian's room. It is, in fact, only a small branch library treated much as is a shut-up shop, to be locked up at night and left, and controlled only by the small staff which would work in the lending library. On the first floor two rooms have been given up to the separate use of boys and ladies, though this has become uncommon of late in such very small establishments, it being thought better to mix the readers and keep all under such general supervision as to prevent annoyance. It is generally thought better to keep boys out of such a place than to provide a special playroom for them—for that is what such a room is likely to become, especially when it is in an

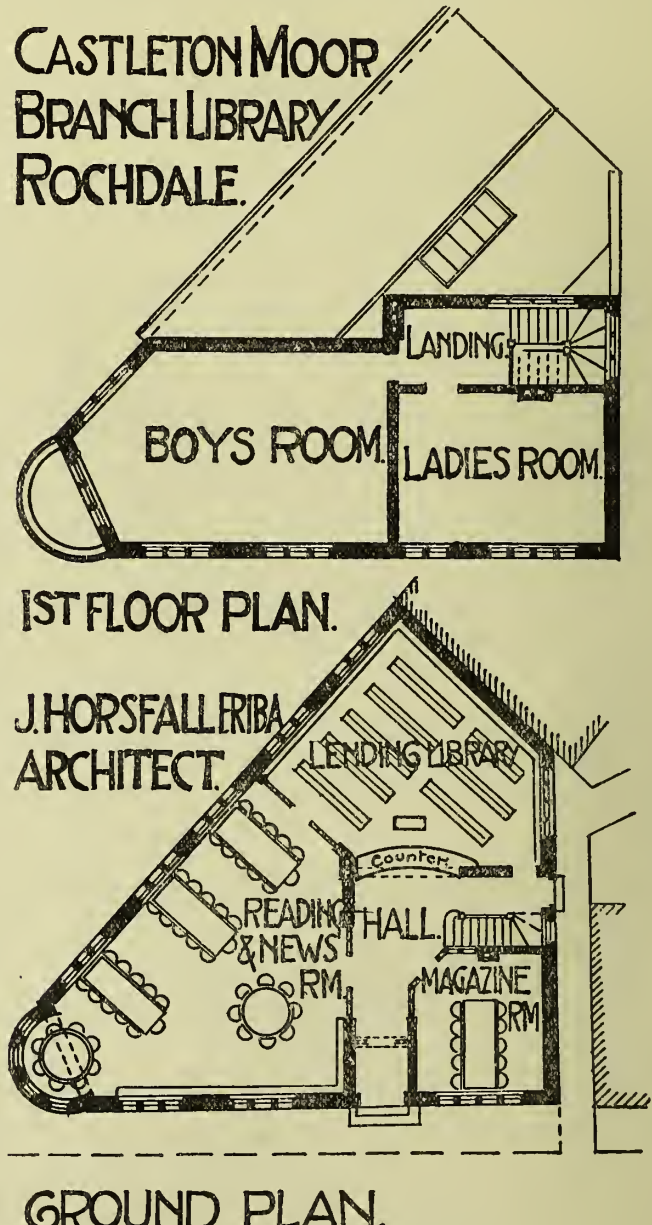


Fig. 6.

out-of-the-way spot,—and to devote the space thus made available to reference use. These matters, however, are not always under the architect's control, but are generally settled by the building committee.

It will be seen that in all these plans there is compactness and directness, naturally combined with general accessibility to all parts both by the staff and by the public. In fact, they all illustrate different

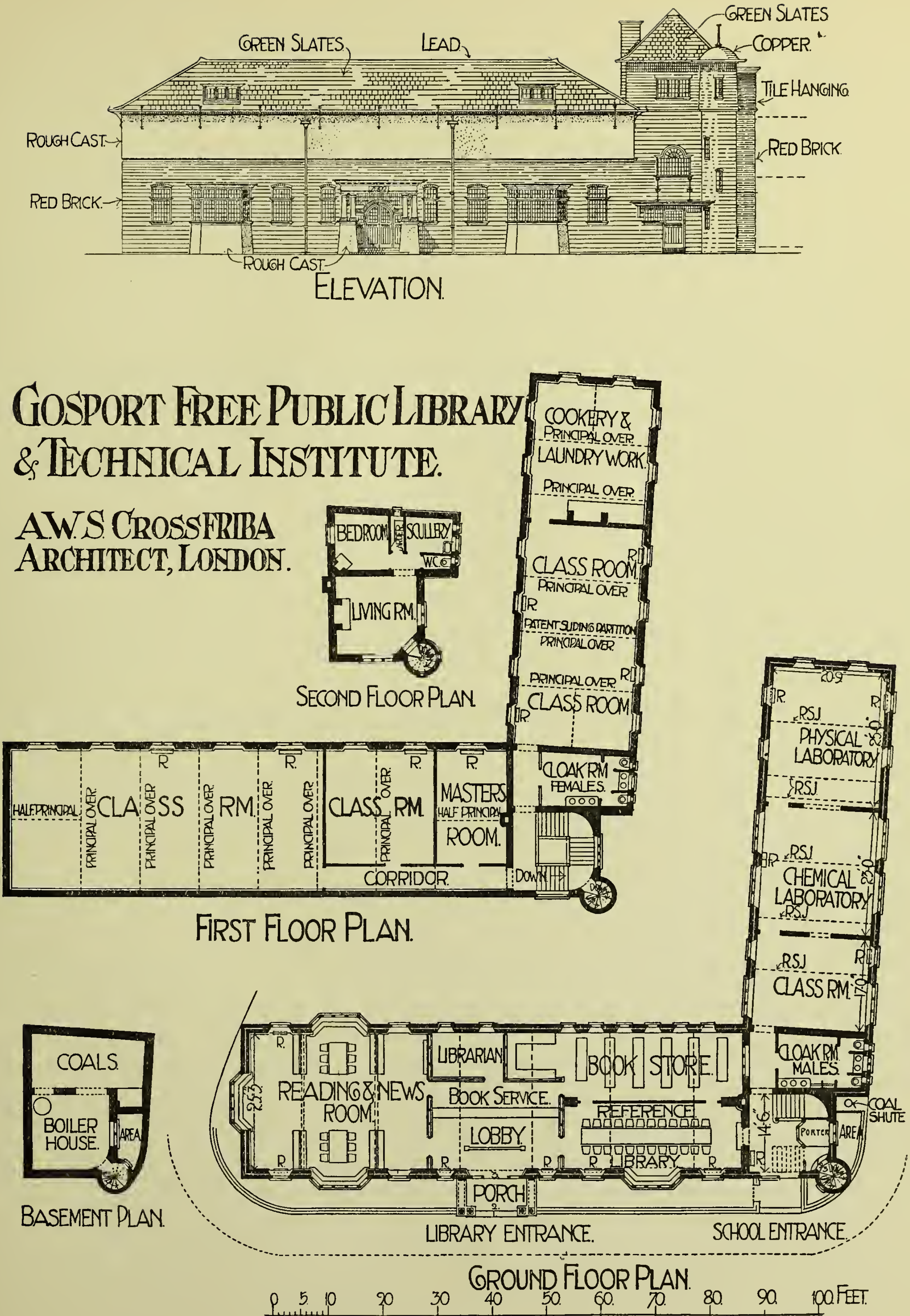

GREEN SLATES
LEAD
GREEN SLATES
COPPER
TILE HANGING
ROUGH CAST
RED BRICK
RED BRICK
ROUGH CAST
ELEVATION.
GOSPORT FREE PUBLIC LIBRARY
& TECHNICAL INSTITUTE.
A.W.S. CROSS F.R.I.B.A
ARCHITECT, LONDON.
BEDROOM
SCULLERY
W.C.
LIVING RM.
SECOND FLOOR PLAN.
COOKERY & LAUNDRY WORK.
PRINCIPAL OVER.
CLASS ROOM.
PATENT SLIDING PARTITION
CLASS ROOM
CLOAK RM FEMALES.
HALF PRINCIPAL
PRINCIPAL OVER
CLASS RM.
MASTERS ROOM.
HALF PRINCIPAL
CORRIDOR.
DOWN
FIRST FLOOR PLAN.
PHYSICAL LABORATORY
CHEMICAL LABORATORY
R.S.J.
CLASS RM.
CLOAK RM. MALES.
COAL SHUTE
PORTER
AREA
COALS.
BOILER HOUSE.
AREA
BASEMENT PLAN.
READING & NEWS ROOM
LIBRARIAN
BOOK STORE.
BOOK SERVICE.
LOBBY.
REFERENCE LIBRARY.
PORCH
LIBRARY ENTRANCE.
SCHOOL ENTRANCE.
GROUND FLOOR PLAN.
0 5 10 20 30 40 50 60 70 80 90 100 FEET.

FIG. 7.

examples of a very similar type, so far as the circumstances of the various sites have permitted.

The Gosport Free Library and Technical Institute, by Mr. A. W. S. Cross, F.R.I.B.A., illustrated in Fig. 7, is a compound building of which the library forms only a comparatively small portion. The borrowers' counter is immediately opposite the entrance, from which it is divided by a light screen ; but the attendants do not find their books immediately behind the counter, but in a distinct bookstore which is planned so as rather more readily to serve the reference library. There is a small librarian's office at the back of the book service, and from this all departments are readily accessible, the reading-room being largely devoted to newspaper stands. The reference library is evidently intended to be used principally by the students of the Technical Institute, into the entrance hall of which it has means of access through a communicating door. The school is planned to some extent on the principles laid down in the last volume, but upon the ground floor the two laboratories and the adjacent classroom open out of one another. The first floor is entirely given up to the school, and contains a series of large classrooms and a room devoted to cookery and laundry work. The plan is L-shaped and well lighted from both sides. Adjacent to the main staircase of the school is a small winding stair, which reaches down to the boiler house and up to quite a small residence for the caretaker on the second floor, about the least possible accommodation being provided for him. The elevation suggests a well thought-out scheme of colour, while it is designed in the simple, old-fashioned style which at the present time is affected by a good many architects.

CHAPTER II

BATHS

Baths, like libraries, are now very generally provided by public bodies, it having been found that they are almost necessities, while they cannot be worked privately at a profit. The only places in which large baths are provided by private means are schools, a bathing establishment being found in all the larger boys' boarding schools, and sometimes in the large girls' schools. Where education of an elementary kind is undertaken by public bodies, however, the scholars usually make use of the public municipal baths, having the right of entry at certain hours when they are not much used by adult bathers.

A simple bath, such as is usually provided in a school, is shown in Fig. 8, which illustrates the bath and library block recently added to Sandroyd School, designed by Messrs Treadwell & Martin, of which the main building was illustrated in Volume III. There is a corridor connecting the entrance to this block with that for boys to the chapel and the school, under command of the window to the assistant masters' room. Opposite the corridor is the door into a single large room which serves as a library. The furniture is not shown, but as a general rule in a school library the books are arranged in shelves round the walls and on stands projecting out from the back wall, while there are plenty of tables occupying the body of the room, and placed in front of the windows, at which boys can read or study. Sometimes the standing book shelves are arranged sufficiently wide apart to form recesses, in each of which a table can be placed for the use of boys who desire unusual quiet.

A long, narrow changing room is interposed between the library and the swimming bath. This room has a seat right round it against the wall, and a stand down the centre for clothes. The room is top-lighted above the clothes stand. Out of the changing room there is a central door which leads straight to one end of the swimming bath,—a long tank graduated in depth, shallow enough at one end for quite little boys to stand in it, while deep enough at the other for a plunge to be taken there by the elder boys. It has a passage way all round it, but otherwise the building is quite clear, save that at the end nearest the changing room there are a few shower baths, which can be indulged in either before or after the swim, or independently of it. For this purpose they are most valuable to boys after playing football or some other game involving heavy exercise, the changing room being used not only for bathing, but when changing clothes for outdoor games.

Behind the swimming bath, but not connected with it in any way, is a most valuable adjunct to a school, in the form of a boiler house and a properly arranged laundry, each distinct. A range of three rooms for boilers, dynamos, and accumulators provides all that is necessary for heating and lighting the building. On plan they do not look to be well lighted, but, as they are only one storey high, top light is of course used to a large extent. These three rooms are grouped together so as to be under the control of one man, but are detached from all others. Behind them is a wash-house where the washing would be done, utilising hot water and steam from the boilers. The arrangement, like that of an hospital laundry, is such that the clothes enter the wash-house from the outer door, and pass thence to an ironing room, which has a large drying cupboard in it where clothes are dried by means of hot air. The finished and dried clothes are eventually passed out of the ironing room by another door to the school. A living room for the staff attached to the laundry, and a coal store, are also provided in this annexe.

An entirely different class of bath, but yet one which is without great complexity, is the new public bath at the Broadway, Tooting, for the Wandsworth Borough Council, designed by Messrs. Druerry & Dolby. This, being erected on a small site and in an exceedingly populous neighbourhood, has been planned for shower baths for men and slipper baths for women, without any swimming bath. It is a very general experience that if an establishment has both slipper and spray baths provided, either one or the other will be used entirely, and as people are at present more accustomed to slipper than to spray baths, the former are more likely to be used. Where the available ground is limited in area, however, the spray bath has many advantages, for it occupies less space and is taken much more rapidly ; while it can also be provided at a cheaper rate. On the ground floor (see Fig. 9) are arranged the men's baths, those for the first and for the second class being in parallel ranges, all top lighted and connected only by a pass-door for the use of the attendant. A corridor runs down the centre of each range, with baths opening out of it to right and left, this being the usual arrangement. It might perhaps be more accurate in this instance, however, to say that

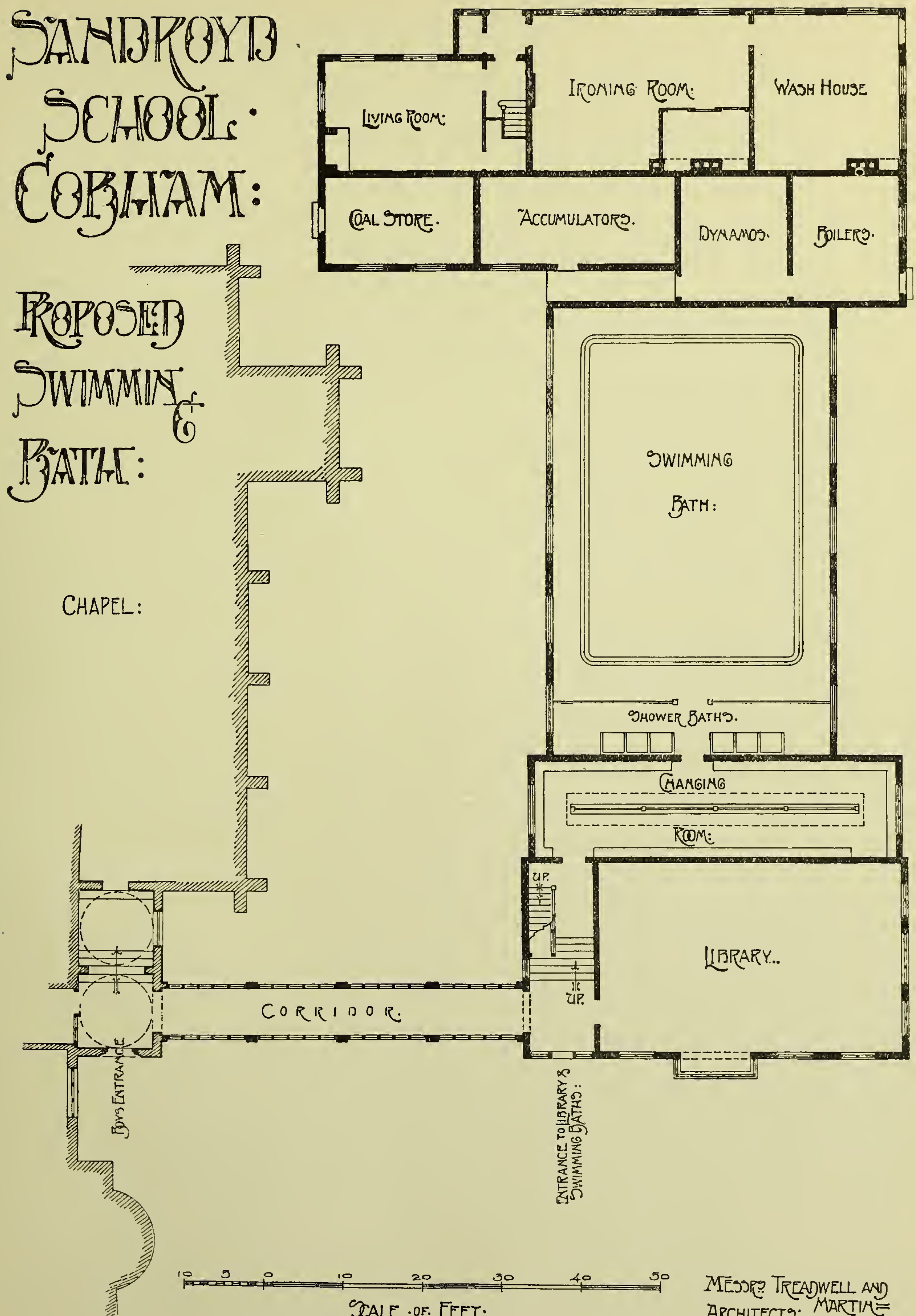

SANDROYD SCHOOL: COBHAM:
PROPOSED SWIMMING & BATH:
LIVING ROOM:
IRONING ROOM:
WASH HOUSE
COAL STORE.
ACCUMULATORS.
DYNAMOS.
BOILERS.
SWIMMING BATH:
CHAPEL:
SHOWER BATHS.
CHANGING ROOM:
UP.
LIBRARY..
UP.
CORRIDOR.
BOYS ENTRANCE
ENTRANCE TO LIBRARY & SWIMMING BATHS:
10 5 0 10 20 30 40 50
SCALE OF FEET.
MESSRS TREADWELL AND MARTIN ARCHITECTS:

FIG. 8.

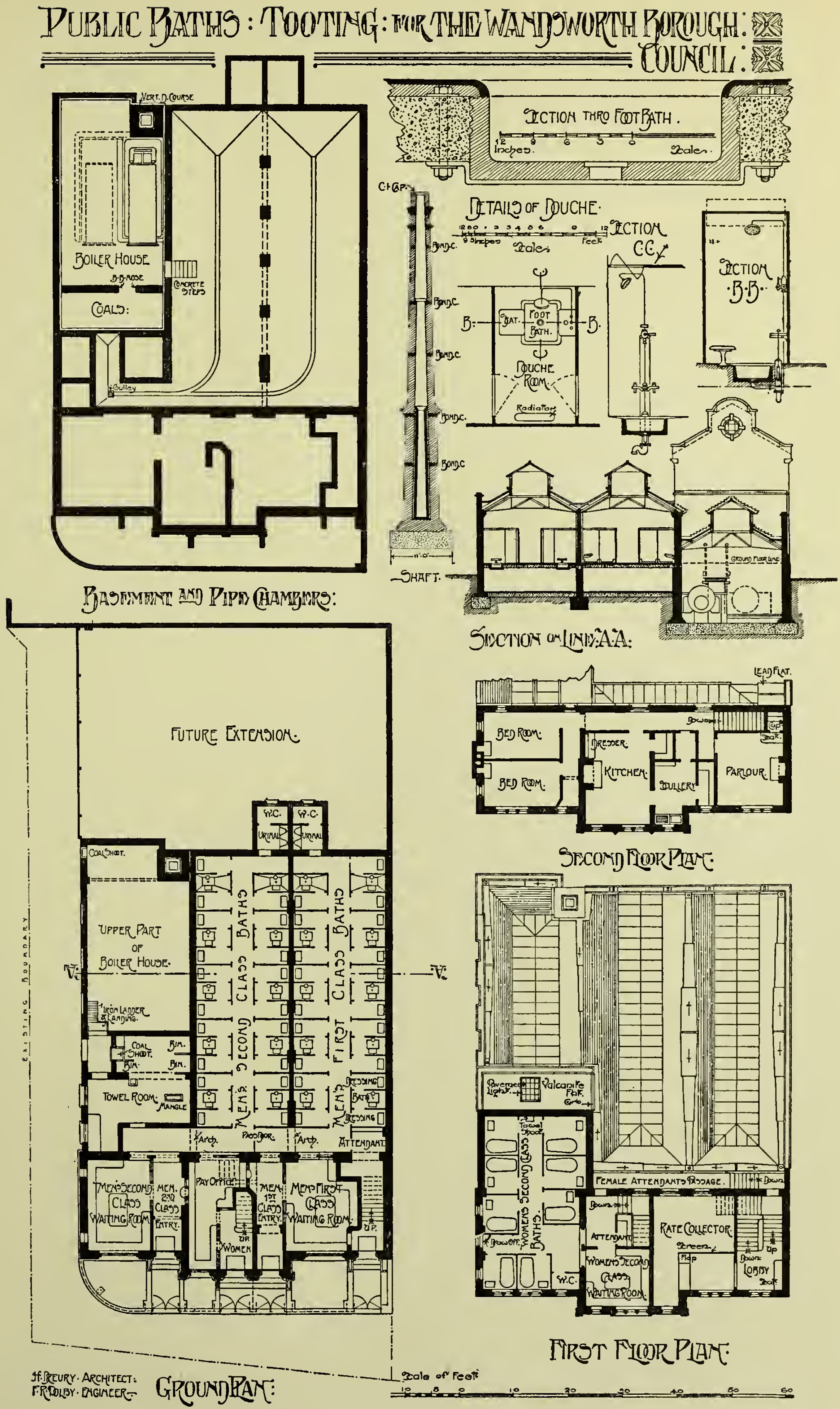

PUBLIC BATHS : TOOTING : FOR THE WANDSWORTH BOROUGH : COUNCIL :
SECTION THRO FOOT BATH.
DETAILS OF DOUCHE.
SECTION C.C.
SECTION B.B.
FOOT BATH.
DOUCHE ROOM.
SHAFT.
BOILER HOUSE
COALS
CONCRETE STEPS
BASEMENT AND PIPE CHAMBERS:
SECTION ON LINE A.A.
SECOND FLOOR PLAN:
BED ROOM.
KITCHEN.
PARLOUR.
SCULLERY
FUTURE EXTENSION.
UPPER PART OF BOILER HOUSE.
TOWEL ROOM.
MENS SECOND CLASS BATHS
MENS FIRST CLASS BATHS
MENS SECOND CLASS WAITING ROOM
MENS FIRST CLASS WAITING ROOM
PAY OFFICE
WOMENS SECOND CLASS BATHS
WOMENS SECOND CLASS WAITING ROOM.
RATE COLLECTOR.
LOBBY
FEMALE ATTENDANTS PASSAGE.
FIRST FLOOR PLAN:
EXISTING BOUNDARY
H. BEURY. ARCHITECT:
F. R. DOLBY. ENGINEER.
GROUND PLAN:
Scale of Feet

FIG. 9.

the dressing-rooms are opened out of the corridor and that the baths are reached from the dressing-rooms; for each bath is provided with two dressing-rooms. This is done for rapidity of service, so that one man can be undressing while another is having his bath, and can then pass into the bathroom while the first man is dressing, thus securing constant use of the bath. By this means the average time occupied by each bather is not much more than six minutes, as compared with an average of half an hour for slipper baths with single dressing-rooms.

A great deal of attention has been paid to the entrances, as is always essential in an establishment of this kind, so that the attendant who is in the pay office may serve all who enter, whether they are going to use the first or the second-class men's baths or the women's baths. It will be noticed that all the entrances are in the front, those for the first-class men's baths and for the women's baths being in the centre, the women going upstairs directly after taking their necessary ticket at the pay office, and the men passing down a short entry to the ticket hatch. They can then either walk directly into the bath corridor or, if all the baths are full, may stay for a while in a large waiting-room, which is entered both from the entrance passage and from the bath corridor, and is close to a small attendant's room. The second-class men's bath is entered on the left-hand side of the front in a very similar manner, being also served from the pay office, and having a separate waiting-room.

The women, on arriving at the first floor, find a small waiting-room before the entrance to the actual baths, which in their case are slipper baths,—or in other words the ordinary baths found in houses. In such a district as Tooting it is not necessary to provide first-class women's baths, as the better class houses have private baths in them; but the second-class slipper baths are very largely used by women, who bring families of small children to give them their weekly tub. There is generally a considerable run upon such baths as these, and the waiting-room, therefore, is a great convenience.

A detail is noticeable here which is always of very considerable importance in bath planning, and that is the towel-shoot. This should be, as it is in this case, absolutely vertical, and should lead directly into a towel-room below, which may possibly be part of a laundry attached to the baths. This towel-room should be equally accessible by shoots or doors from all parts. It will be noticed that at Tooting it can be reached by a door on the ground floor from the men's bathing establishment. It is not intended to wash the towels at Tooting, though a mangle is provided if it be only necessary to smooth them out; for the Council possesses a larger bath at Wandsworth where there is a proper laundry, and it would be quite a small matter to carry the towels there for washing. The towel-room contains a series of bins for storage; but it must not be imagined that the coal-shoot is open to the towel-room, as it might at first sight appear to be. On the other hand, it discharges directly into a coal-store in the basement, where is situated a large boiler house, to be provided with one boiler at the outset, situated close to the chimney shaft, and having a bedding provided for a second boiler to be inserted later on. The whole space underneath the men's baths is excavated and can be reached from the boiler house, and it is in this space where all the necessary piping, electric wires, etc. would be carried, as can be seen clearly on the section.

On the first floor, in addition to the women's baths, there is a small office for the use of the Rate Collector, this being a convenient centre for him, though not one of the principal offices in the district.

The second floor is given up to a house for the use of the caretaker, containing a kitchen, scullery, parlour and two bedrooms, it being of course possible for the parlour to be used as a third bedroom in case of need.

The details of the douche or spray bath (Fig. 9) may be of some interest. There is a square foot bath sunk in the floor, with a seat on one side of it, and a stand on the other on which are arranged the various taps for supplying water, either to the foot bath only or to the shower. This is arranged so as to spray directly upon the head of a person standing in the foot bath, and can be controlled so as to give any temperature of water which may be required; but a clever arrangement provides that excessive heat such as would be dangerous can never be attained. Each bath room is warmed by a radiator.

A much more complete and complex building is that erected for the public baths in the Old Kent Road (see Figs. 10 and 11, introduced here by permission of *The Builder*), from the designs of Mr. E. Harding Payne. It was there desired to have two large swimming baths, and first, second, and third-class slipper baths both for men and for women, as well as a public wash-house and a Turkish bath; while the principal swimming bath had to be arranged for swimming contests, and so that it could be used as a hall for entertainments during the winter time. A complicated problem like this is always difficult to solve, calling for a great deal of skill upon the part of the designer, and this is particularly the case when, as in this instance, the site is irregular.

As a matter of necessity the swimming baths are placed upon the ground floor, and the first-class bath occupies the principal position, flanking Marlborough Road. Its main axial entrance is planned rather for its use as a public hall than for bathing purposes, there being a pay-box dividing two streams of entrants, who pass direct on to what would be a level floor at the level of the passage round the bath. At the side of this entrance is another which leads to the gallery, while a second gallery entrance, or more probably an exit only, is obtained from Marlborough Road. These

entrances, as has been said, are all intended for use only when the bath is converted into an entertainment hall, as also are three side exits from the bath hall direct to Marlborough Road, two of which can only be reached by the removal of some of the temporary dressing boxes which are introduced during the bathing season. These many direct exits, like the direct axial entry, are always necessary in places used for entertainment purposes, to enable them to be emptied with extreme rapidity in case of emergency.

At the farther extremity of the bath hall are two rooms for artistes, from which the stage can be entered, and having a corridor entry at the back which is used in the ordinary course of events as the entrance to the rooms, which, like the women's first-class waiting-room, is entered from a women's hall reached by a separate women's entrance. One ticket-office serves both the men's and women's entrances, and also that for the Turkish baths, the staircase to which can be shut off from either men or women, as may be desired, at any time. There are a large number of women's first and second-class slipper baths, and a few spray baths are also provided for them, almost all being top lighted.

The first floor is given up to the gallery of the first-class swimming bath and to a great number of slipper baths for men, who in the Old Kent Road are provided with these in preference to spray baths, though a

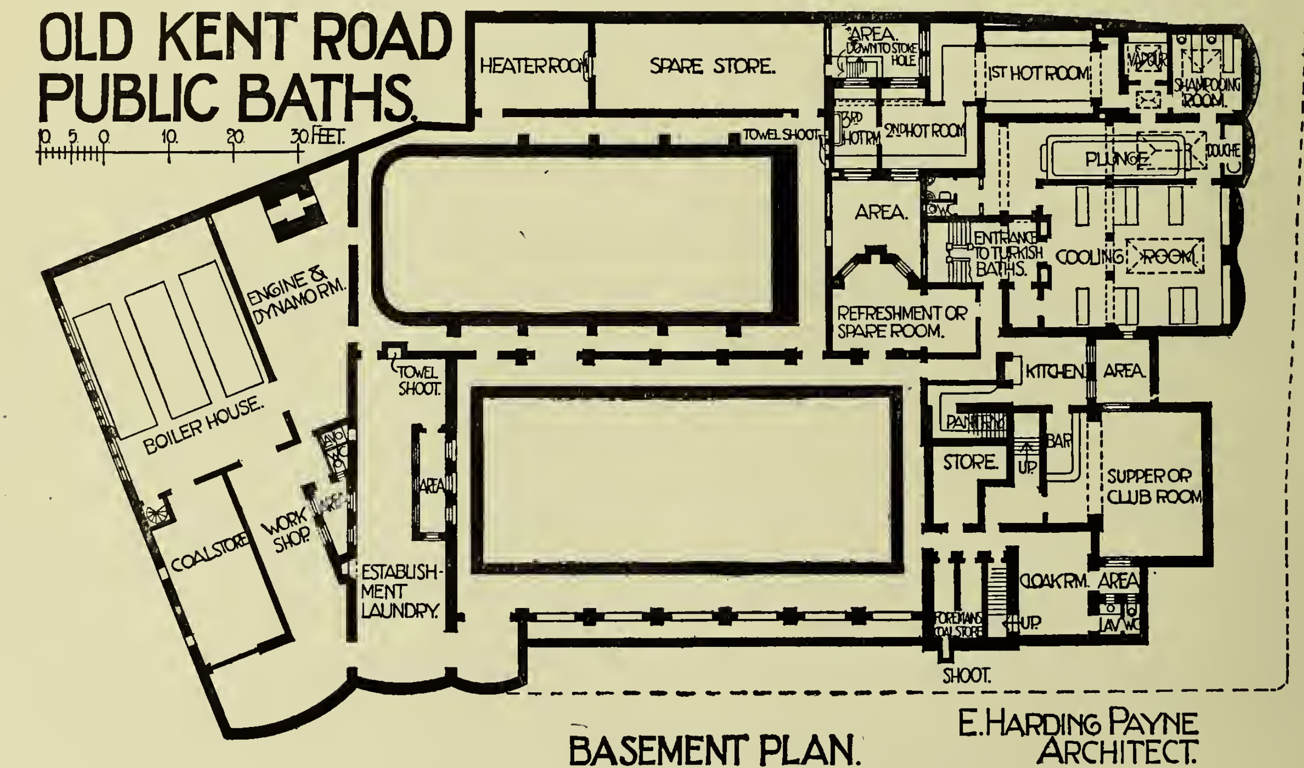


Fig. 10.

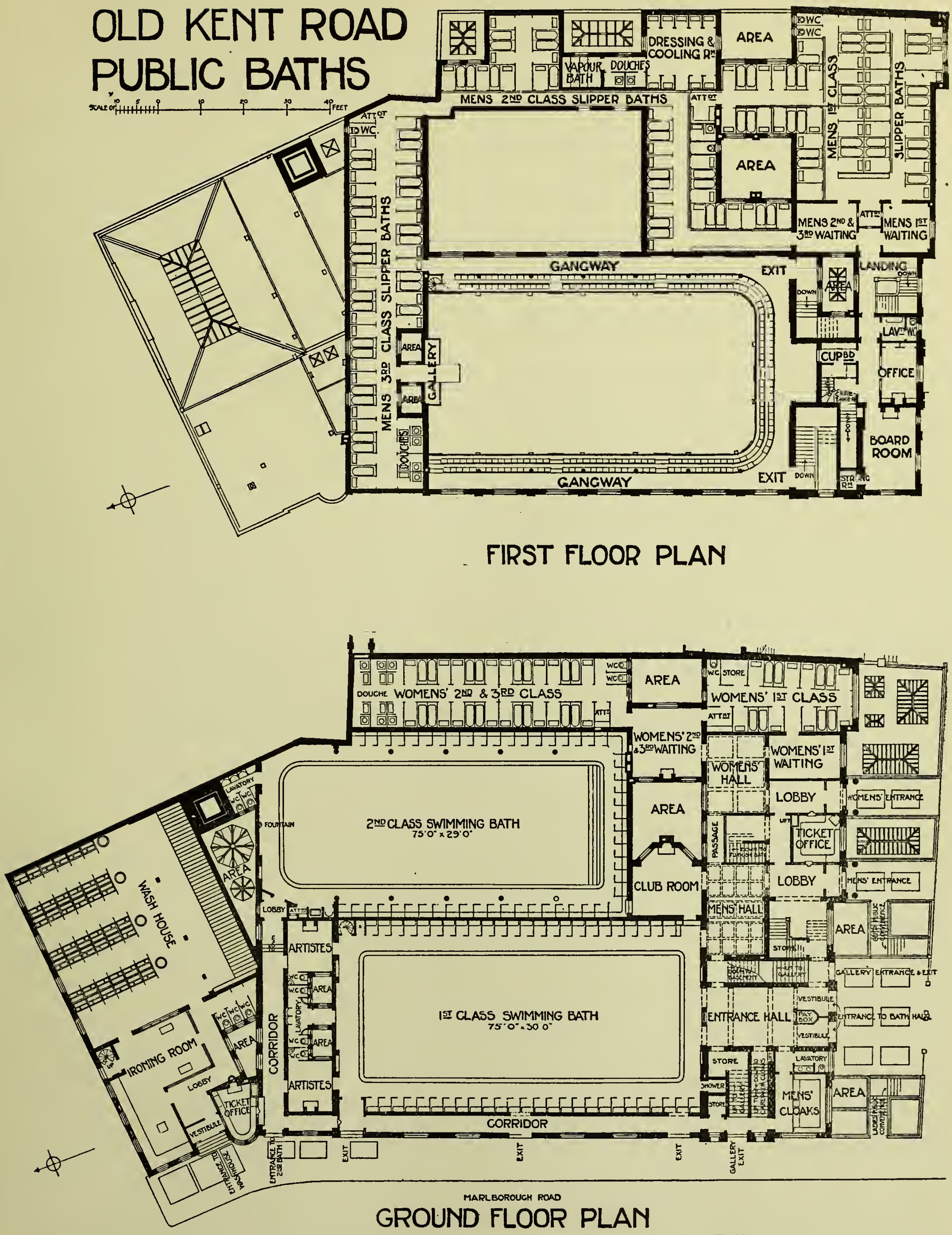
second-class swimming bath, served by a ticket office which also serves the wash-house.

This is the only entrance to the second-class swimming bath, except a door from a general club-room, which communicates with both swimming baths and also with a men's hall, reached from a lobby and the men's entrance in the front. It may be taken that, as a rule, the club-room would only serve the first-class bath, but that the second-class bath might be used for club purposes in the winter and off season, when the first-class bath would be given up to entertainments. It may be noticed that both baths are practically the same size.

The second-class bath would be reserved for women on certain days of the week, there being a doorway to it from the women's second and third-class waiting small department is devoted to douches and vapour baths, with dressing and cooling rooms *en suite*.

At the rear of the site, on the ground floor, is a fully equipped wash-house. The washing troughs are arranged in bays, standing out from the wall very much as do the book stands in public libraries, while along the back of the wash-house are a series of drying cupboards, the racks running on wheels, and being somewhat like towel horses, being drawn out for the clothes to be hung on them, and then pushed back into the drying cupboard for warm air to play round them until the clothes are dried. They can then be taken out to an ironing-room, which is provided with long ironing tables and the necessary mangles. The range of boilers will also be seen in the wash-house.

Perhaps the most important part of this plan is the



E. Harding Payne, Architect.

FIG. 11

basement (see Fig. 11), which, as usual, contains the boiler-house, so contrived that the boilers can be removed for renewal when necessary without doing structural damage to the building, and an engine and dynamo-room, besides an establishment laundry into which a towel-shoot from the upper floors discharges. From this portion of the building there are subways carried round below all the baths and dressing-rooms, for the usual purpose of conveying pipes and wires, while in the front there is a complete Turkish bath, reached by a stairway close to the ticket-office on the ground floor. This establishment consists of a cooling-room, a small plunge, a douche, a shampooing-room, vapour-room, and three hot-rooms, arranged in sequence for various temperatures. It is perhaps the first time that a complete Turkish bath like this has been provided by a municipality, and in this case it is found in a district which is somewhat crowded and where it is greatly needed, and yet where a separate establishment would not be likely to pay expenses. It is, of course, small when compared with some of the west-end baths of this description, but it is complete in all necessaries.

It is interesting to compare an English public bath like this with that erected in the city of New York by Messrs. Aiken & Brunner (see Fig. 12, which is a reprint from the *Builder's Journal*), which is a combination of shower baths and swimming baths. The arrangement provides for 75 baths for men and 42 for women, it being an essential part of the scheme that every bather must first use the shower bath before being admitted to the plunge. The men's swimming pool occupies the centre of the site, and is only to be reached by a run of some considerable length from the shower baths and dressing boxes; but on the women's side the more convenient arrangement is followed of placing the shower baths round the pool. Around the edge of each swimming bath there is an overflow trough. It being expected that there would be great crowds using the baths, each side is provided with separate entrances and exits. The waiting-halls for men and women are in the centre of the front, separated only by the ticket-office, which serves both; and it will be noticed that the external doors are of the four-way revolving character. The exits are quite independent, so that the persons leaving the bath do not conflict with those entering, an arrangement which in England might occasionally be inconvenient, especially if the bathers had brought with them portable property, such as bicycles, which they wished to

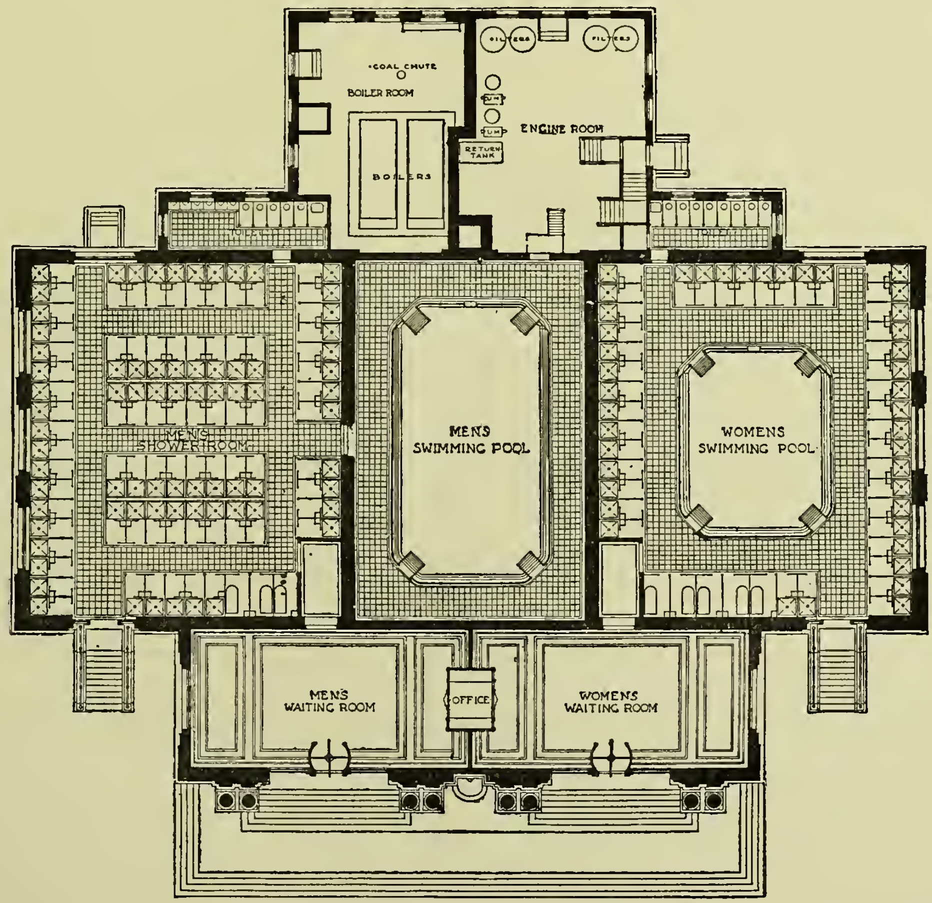


Fig. 12.—Public Baths, New York City. *Aiken & Brunner, Architects.*

take up again before leaving. The engine and boiler-rooms are arranged at the back, and not in a basement. The elevation (Fig. 13) suggests a somewhat handsome building in a single storey.

An entirely different class of bathing establishment is that for medicated baths attended by the wealthy classes as much for what is known as "taking the waters" as for bathing. A bath of this type is always of a more architectural character, designed to a great extent for display and attraction, and generally on a fine open site. Such an establishment, designed by Mr. W. G. Kerby, is illustrated in Plate I. It is approached by a large flight of stairs leading to an open colonnade, providing a sheltered lounge in hot weather. From this the bath-house proper is entered, a large hall occupying the centre of the building, with alcoves on either side for "taking the waters." In these alcoves there are large bars, and lifts are shown for the supply of glasses, etc. from the basement. The central hall, and the alcoves also, are top lighted, and round the whole of this space passes a broad corridor, the women's bath proper being arranged on one side of the axis of the building, and the men's baths on the other. These are exactly similar one to the other. Each is provided with a series of cloak-rooms in the front, and with a large lift, up which persons can be carried in bath chairs from the ground level. A complete wing on each side is given up to slipper and medicated baths, which open out of a long hall; and another projecting wing at the back is devoted to a swimming bath, designed as a handsome apartment. Between the two swimming baths there is a long narrow open court, with flower beds and a fountain. In fact, the whole place is arranged on sumptuous lines. All the lighting and heating arrangements are in a basement, the plan of which has not been given.

It will be noticed that the plan is entirely different to anything hitherto considered in this or the previous volumes of the present book. In all subjects hitherto considered the planning has been upon what is known as the Gothic system, irregular and depending upon the requirements of the building. This plan is on what is more generally known as the Classic system, and its main object is to produce a building of highly architectural character, both internally and externally. It is arranged upon an axis, the entrance being axial and the entrance corridor also, and when the central hall is reached a cross axis dominates the wings. For formal buildings this system of planning is almost essential, leading to symmetry of arrangement and the possibility of good architectural effects, but it is more applicable to larger than to small buildings. The object is to obtain a general simplicity and easy supervision, combined with symmetry and the provision of vistas. In such an example as that now before us all this is comparatively easy, but upon a restricted site or one of awkward shape it is sometimes exceedingly difficult to lay down the primary axis satisfactorily, and afterwards to determine the minor axis, which may traverse the major axis or may diverge from it from some meeting place, such as a central hall may form. Several other examples of this kind of treatment will be given later on.

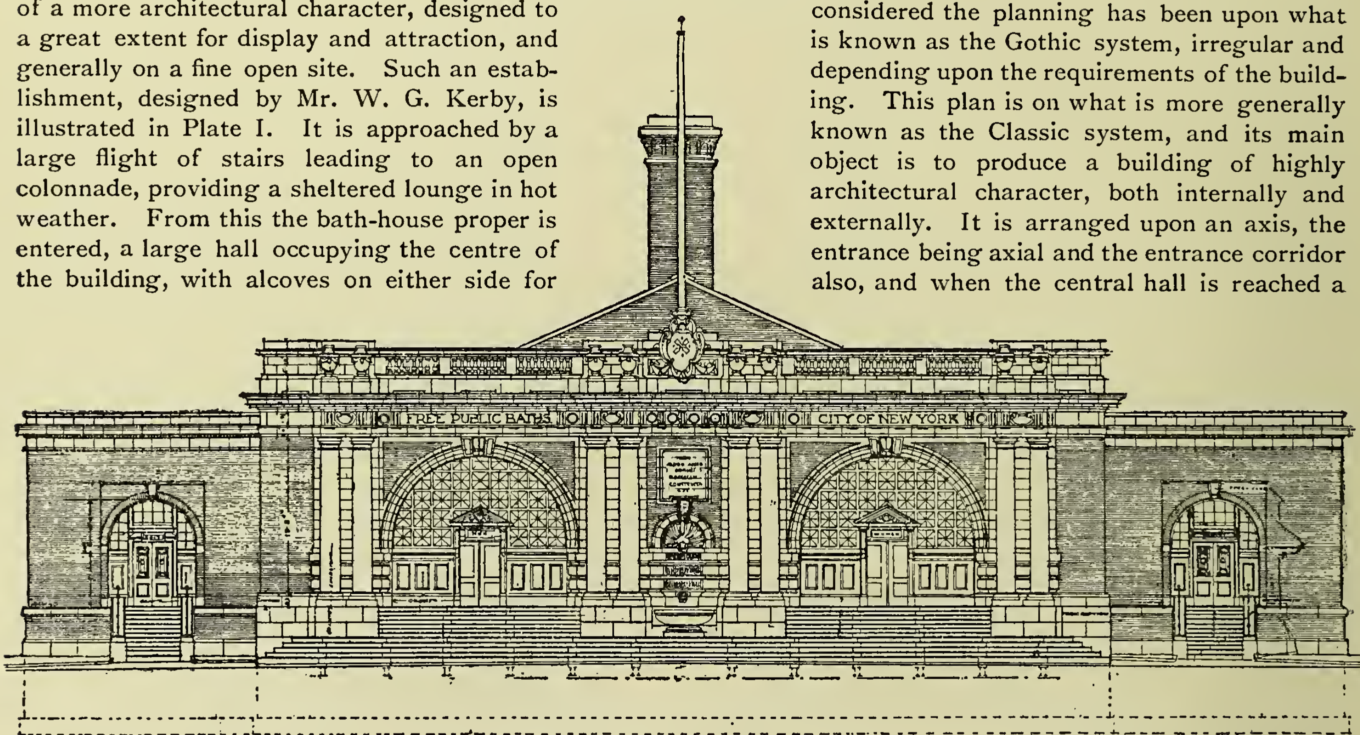
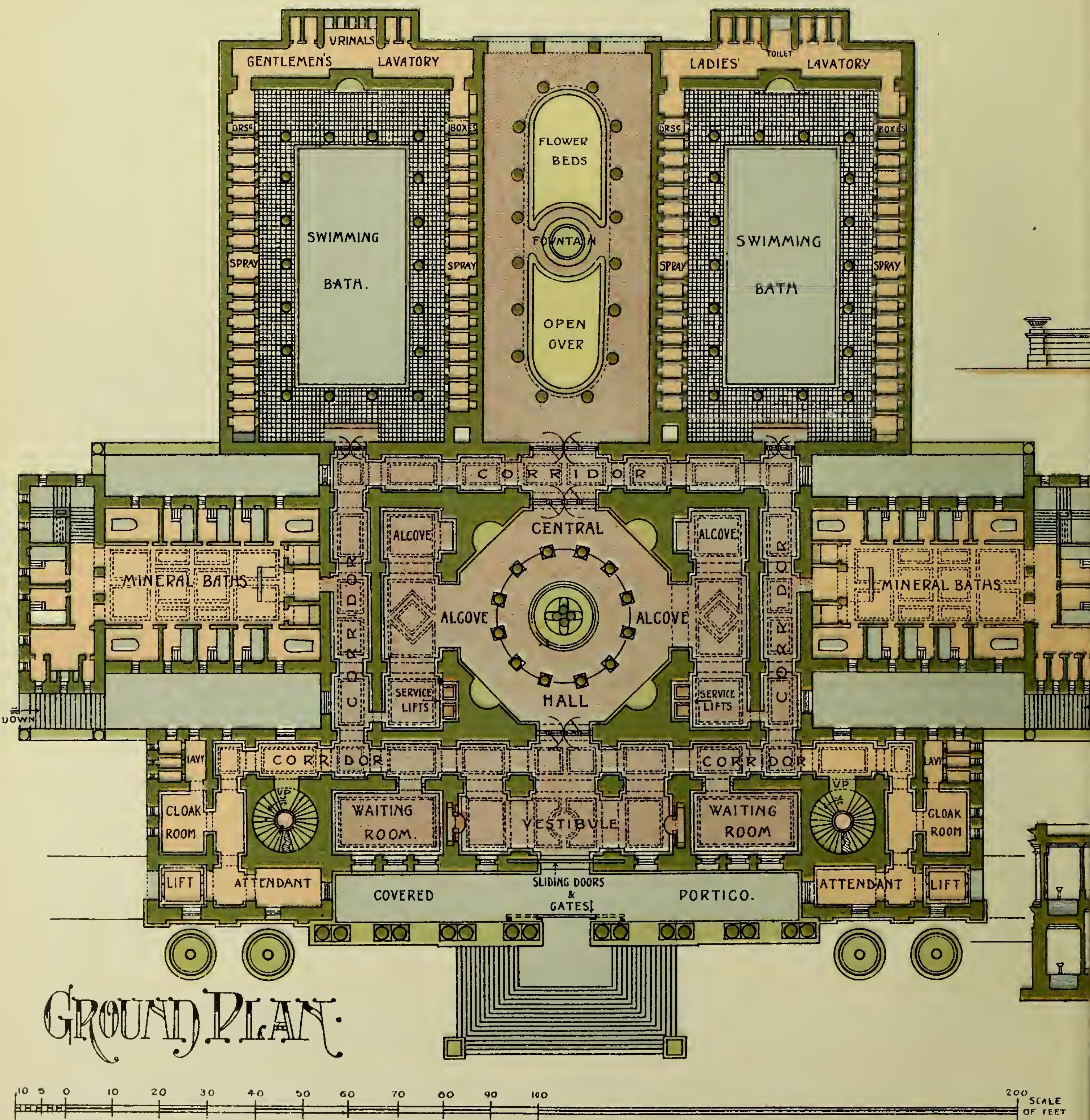


FIG. 13.—Public Baths New York City. *Aiken & Brunner, Architects.*

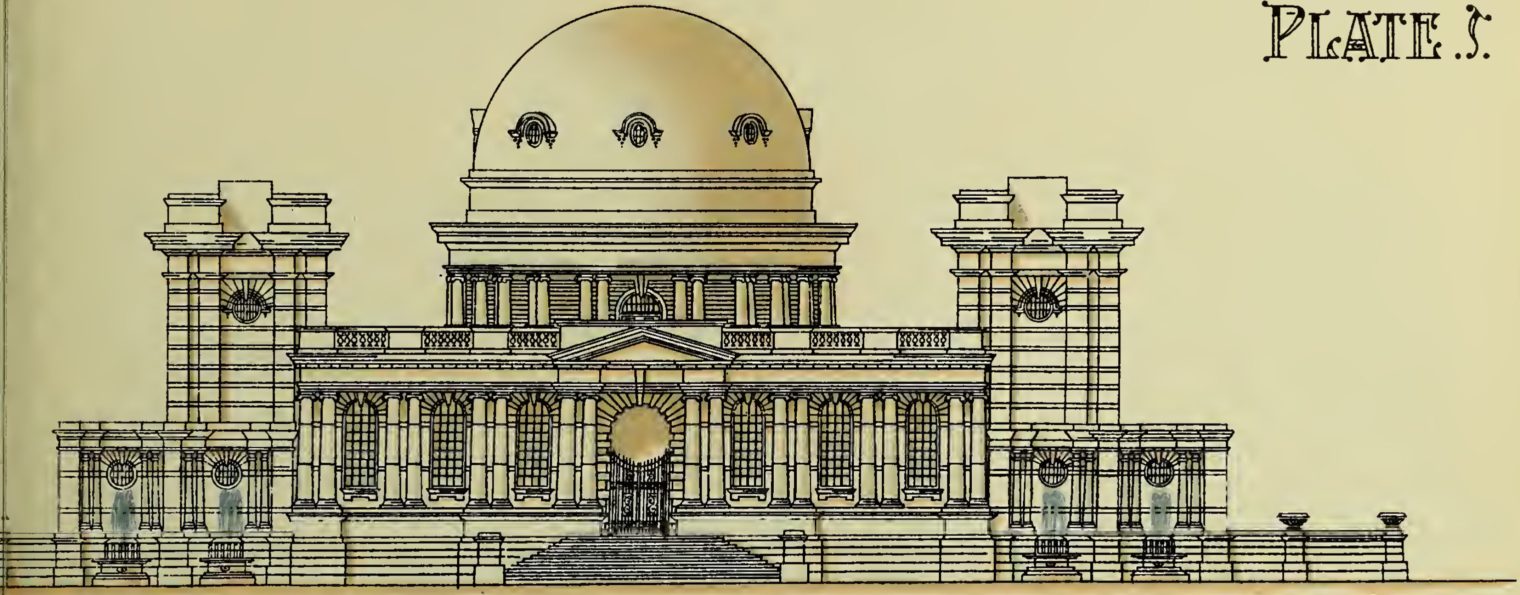
DESIGN FOR MINERAL SPRING A



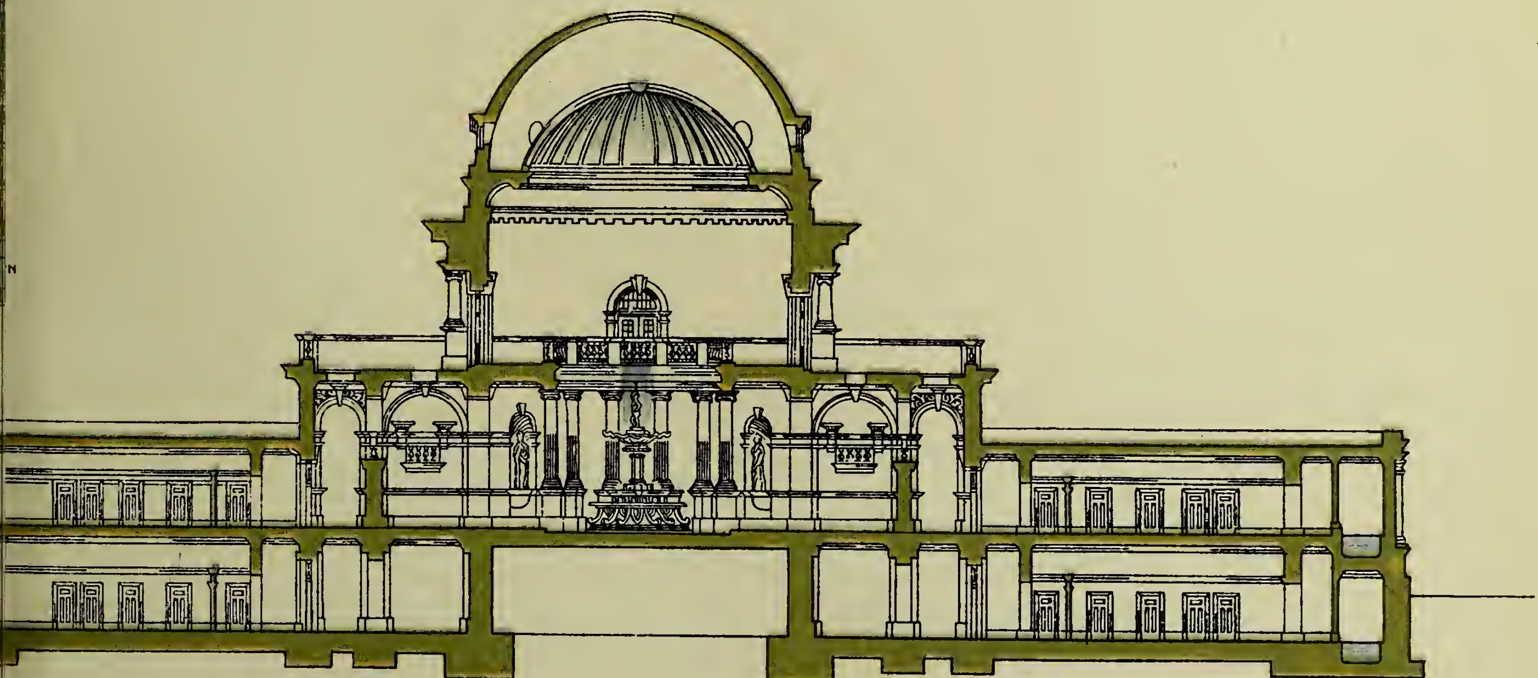
GROUND PLAN.

ND SWIMMING BATHS AT A SPA :

PLATE 5.



FRONT ELEVATION:



CROSS SECTION:

W. G. KERBY, ARCHITECT.

CHAPTER III

SMALL MUNICIPAL BUILDINGS

The buildings provided for municipal purposes are usually of a complex character, although in this respect, and in their magnitude also, they vary to a large extent. It is somewhat difficult to discriminate, and to separate them into classes, but for the purpose of convenience the smaller buildings are here being treated first.

Of these a typical example may be found in the municipal buildings of Cowdenbeath in Scotland. A comparatively small township, ruled by a council of but few members, for deliberative purposes, there was therefore only needed a council chamber or a committee room together with town-clerk's offices. It was thought well to accommodate the Burgh Court in the same building, and a couple of additional rooms have been added in case of their being needed in the future, as they very well might be, for the local surveyor. The problem thus set was not a very difficult one to solve, and it has been most satisfactorily met in the design of Mr. T. H. Ure (see Fig. 14). The town-clerk's offices are arranged on the ground floor in the front, and are *en suite*. The town-clerk's private room, and that of his principal clerk, open from a hall which is served by the main entrance, and out of which stairs rise to the first floor. The town-clerk's general office is at the corner of the main and side streets, and adjoining it, entered from the side street, is a waiting-room. Thus any of the general public who may wish to consult the town-clerk or his staff would enter from the side street, while the members of Council would have access to their offices from the private entrance in the main street.

Immediately accessible is the Court room, while the two rooms for the magistrates and their clerks occur behind it, and are reached, at the level of the platform, off the main staircase. In this way the town-clerk could also act as clerk of the court, and in any event communication is easy. An extension of the building down the side street contains a public entrance, together with rooms for the witnesses and what is called a charge-room, though more properly it should be a prisoners' room, in which the prisoners are kept while waiting either before or after trial. This annexe is cut off from the Court room by a ventilated lobby, out of which the dock is directly reached. It will be noticed on reference to the perspective (Plate II.) that the elevation does not in this case suggest the plan, nor grow out of it very naturally, and this is perhaps a pity, as the plan is a perfectly satisfactory one, and could have been treated to produce an elevation with unbroken frontages which would have expressed the meaning of each part.

The council chamber is situated on the first floor, at the junction of the two streets, and is arranged so that the members of the council sit at a U-shaped table, facing inwards towards the principal seats occupied by the mayor and the town-clerk. A committee-room adjoins it, which can be used independently or entered from the council room, while one of the additional rooms is also planned for communication in case of necessity.

Fig. 15 illustrates a somewhat more complicated arrangement, at Erdington. The town is a somewhat larger one, governed by a larger council, and it was desirable that the council chamber and its connected rooms should be arranged to form a handsome suite which could be used for receptions. The general offices, too, were to include a department of some considerable size for the use of the surveyor, while offices were also needed for the rate collector, sanitary inspector, and accountant. Added to this, the public library was to be housed in the same building, while the caretaker was to be given a residence, and stables were to be provided for three horses. The problem thus became one of some complexity, especially as it was desired that the building should be so designed as to allow of extension in the future, both of the library and the municipal offices. The site, like that at Cowdenbeath, was a corner one, while there was ample space around for lighting to be obtained upon all sides. The architect, Mr. G. P. Osborne, F.R.I.B.A., adopted an axial system of planning, but used two independent axes, one being obtained by bisecting the frontage of the library which faces the main road, and the other by bisecting the remainder of the building along the side road after subtracting the portion used as a library. This second axis, indicated on the plan by the letters AB, he again intersected by a minor axis, in the form of a long corridor off which the various municipal offices were entered, while it formed a means of communication between these and the library.

Considering the library first, it will be noticed that its plan conforms with the general arrangement explained in the last chapter with regard to other examples. An axial entrance leads to a large hall, with the borrowers' library directly in front, and the

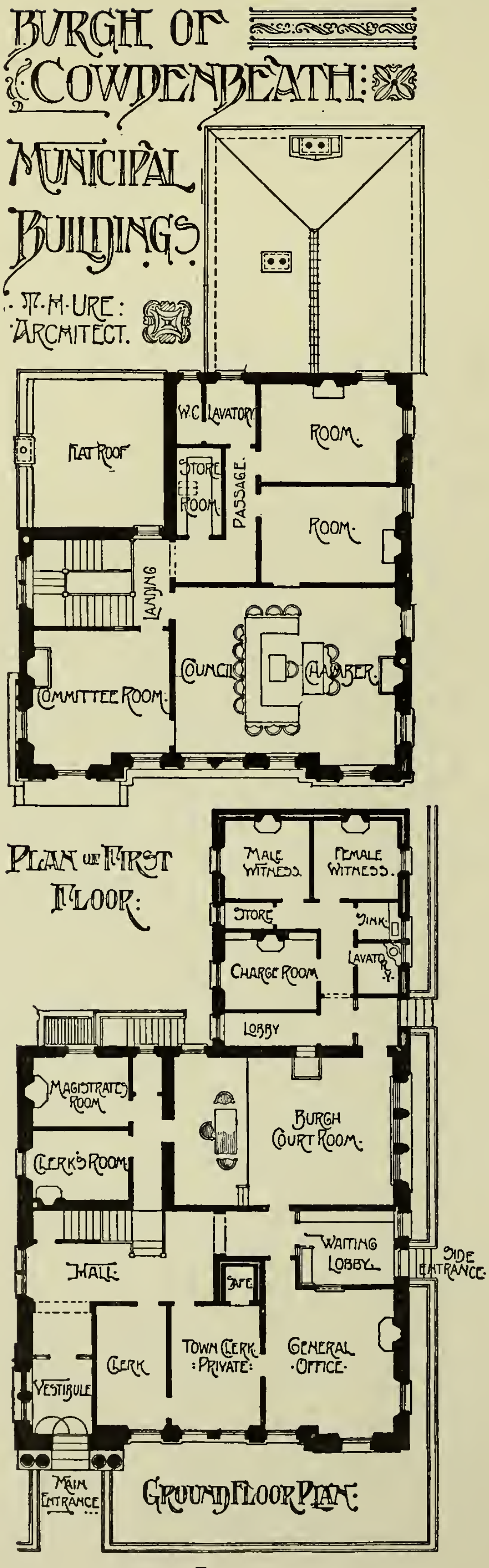


Fig. 14.

newspaper and magazine-rooms to right and left. The librarian's room also has a door from this entrance hall and another into the lending library, while the reference-room is reached through the newspaper-room, and has a counter in the wall separating it from the lending library, so as to enable it to be served with books from there. Glazed partitions are used wherever possible, in order to provide for supervision by the staff and of the public by themselves. There is a staff entrance behind the librarian's room from a large yard entered by gates at the side of the building, and stairs from this entrance lead up to store-rooms and repairing rooms on the first floor, and to a small room entered out of the repairing-room, and devoted to keeping the files of newspapers. There are two doors at the back of the lending library, one of which opens into the extremity of the cross corridor of the municipal offices, and the other into the caretaker's premises, which also communicate with this cross corridor, as does the reference-room as well.

The caretaker's house is larger than is often provided, there being a sitting-room, kitchen, and scullery on the ground floor, and three good bedrooms upstairs over the reference and newspaper-rooms, together with a large linen cupboard and a general store which could be converted into an additional bedroom if necessary.

The municipal offices proper are planned as a self-contained building upon the axial passages already mentioned. There is a carriage entrance contrived for in-and-out passage of vehicles, enabling them to deposit their occupants at the main door, from which a vestibule is entered, overlooked by a small caretaker's office, which also has command of the carriage-way. Passing through two pairs of double doors, a fine hall is reached, with a staircase rising immediately in front of the entrance and well lighted by three staircase windows. The stair is so placed that it does not interfere in any way with the clear passage from end to end of the transverse corridor, out of which, on the left-hand portion, are entered a series of rooms for the accountant and the rate collector, a waiting-room being placed between the two, entered from either. All of these rooms have glazed screens on the corridor side, which thus obtains borrowed light and is to a certain extent supervised. Opposite the rate collector's office is a room for the sanitary inspector and medical officer, from which there is also an exit to the yard through a small lobby, for the use of workmen, who can by that means obtain access to a room set apart for disinfectants yet under the control of the inspector. Along the right-hand portion of the corridor, in relation to the entrance, are arranged the surveyor's rooms, consisting of a general office, a private office, and a large drawing office having a north light, with a strong-room attached to it for the storage of plans. At the end of this corridor there is a workman's entrance, serving a small waiting-room or office and giving access to a pay hatch in the drawing-office. The means by which the surveyor has control

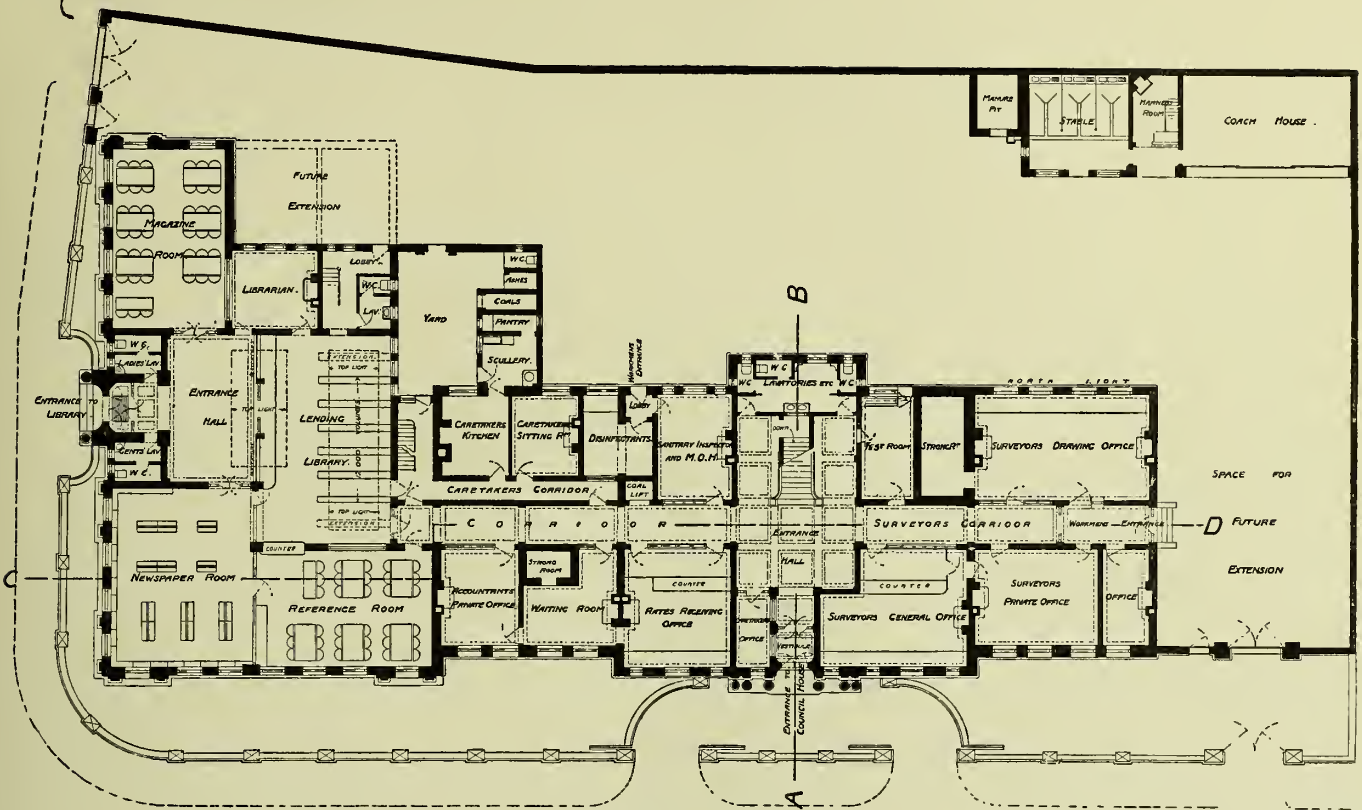
PLATE II.

MUNICIPAL BUILDINGS, COWDENBEATH, SCOTLAND.

[T. HYSLOP URE, ARCHITECT.

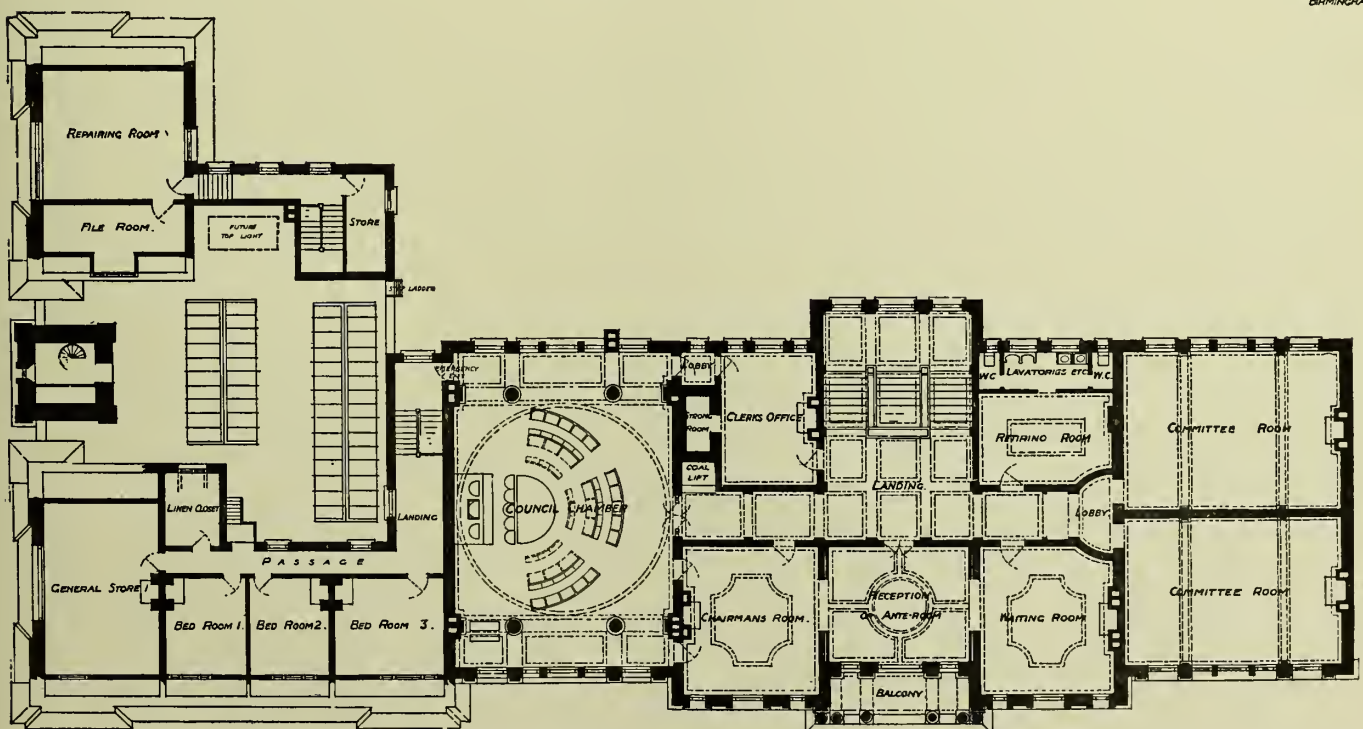
Council House

and Free Library Erdington



Ground Plan

John P Osborne
Architect
95 Colmore Row
Birmingham



First Floor Plan

Fig. 15.

of his staff and ready access both to the main building and the yard is noticeable. The only other room on this floor is a test-room, mainly for the use of the surveyor, for testing materials, and provided with a large sink ; while the lavatories are arranged underneath the staircase.

The axial arrangement of the ground floor also controls the upper floor, which is thus easily planned for architectural effect. The staircase divides at its landing, and returns as two flights to the first-floor level, the town clerk's office being immediately at the top of one of these flights, and so placed as to communicate by means of a lobby with the council chamber, the main entrance to which is at the western end of the transverse corridor. This is a handsome rectangular room, with the central portion reserved for the council, whose seats are arranged in segmental form, while there are aisles on either side. There is an emergency exit, for use in case of fire or panic, to the caretaker's stairs, which would also form his ordinary means of entrance for cleaning. A chairman's room corresponds to the town clerk's office, and is entered directly both from the corridor and the council chamber. A fine reception-room faces the staircase, and there is a waiting-room on its eastern side, which, together with the reception-room, the chairman's room, and the council-room, are *en suite*, so that the building could be used not only for council meetings, but for large entertainments. There are two large committee-rooms at the eastern end of the building, as well as a large robing-room with lavatories. The plan of this floor is simple and dignified, and has evidently controlled that of the floor below. It is a good example of formal planning for a small building, showing how by the use of system an apparently somewhat complicated problem can be made extremely simple.

CHAPTER IV

LAW COURTS

LAW COURTS are buildings which are often included in the same erection as other municipal offices. Occasionally, however, they are independent, and it is perhaps by considering one of these cases that the requirements can be best understood. On the Continent it has been usual to build the Law Courts separately, and to make them important edifices, designed for architectural effect rather than for convenience. In England the tendency has been different, and often the effect has been subordinated to the interests of the plan, while in one notable instance, the Civil Courts in London, an originally good plan was spoilt by a transference of site, and an architecturally beautiful building is admitted to be an inconvenient one.

The principal examples of independent buildings of this character in England are the Manchester Assize Courts, designed by Mr. A. Waterhouse, R.A.; the Birmingham Courts, admitted to be admirably arranged from the designs of Sir Aston Webb, R.A., and Mr. Ingress Bell; the London Criminal Courts, at the Old Bailey, now being built from the designs of Mr. E. Mountford, F.R.I.B.A.; and the Cardiff Law Courts, recently completed by Messrs. Lanchester, Stewart, & Rickards.

These last have the advantage of being placed on an isolated site, so that architectural treatment was not only possible but rendered imperative, while the design was controlled by that of the Town Hall, erected by the same architects close by, and of the same width from north to south. The general arrangement may be seen by the block plan, Fig. 16,[1] the two great buildings being separated by an avenue planted with trees. The site being absolutely limited largely controlled the arrangement, and forced the placing of some of the Courts upon the first floor, which on this account dominated the plan to a great extent. As a matter of convenience, however, it may be well to give the ground floor preference in consideration, but readers will well understand, on comparison, that it was the upper floor which had most to say in the original laying out of the scheme.

(1) Assize Court. (Criminal.)	Sheriff. Deputy Sheriff. Judge. Judge's Marshal. ,, Clerk. Assistant Clerk.	Jury of Twelve. Jurors in waiting.	Bar.	Witnesses and Solicitors. Ushers. Official Reporter. Press.	Prisoners. Warders. Police.	Public.
(2) Assize Court. (Civil.)	As above.	As above.	,,	As above, and Litigants.	None.	,,
(3) Quarter Sessions. (Appeals from police and petty sessions in minor sessions; licensing cases.)	Magistrates, with Chairman of Bench or a Recorder. Clerk.	None.	,,	Applicants. Witnesses and Solicitors.	Prisoners and Police.	,,
(4) Petty Sessions. (County Police Court.)	Magistrates or Stipendiary. Clerk.	None.	,,	Prosecutors. Witnesses and Solicitors. Press.	Prisoners and Police.	,,
(5) Police Court.	Magistrates or Stipendiary. Clerk.	None.	,,	Prosecutors. Witnesses and Solicitors. Press.	Prisoners and Police.	,,
(6) Wreck Court.	Stipendiary. Nautical Assessor. Clerk.	None.	,,	Ship's Officers. Witnesses and Solicitors. Press.	None.	,,
(7) Sheriff's Court. (Compensation cases and assessment of damages.)	Deputy Sheriff. Clerk.	Jury of Twelve.	,,	Plaintiff and Defendant. Witnesses and Solicitors.	,,	,,
(8) County Court.	Judge. Registrar. Clerk.	Jury of Five.	,,	Plaintiff and Defendant. Witnesses and Solicitors.	,,	,,
(9) Coroner's Court.	Coroner.	Jury of Twelve.	,, (Occasionally).	Witnesses.	Occasionally prisoner under escort.	,,

[1] All the illustrations in this Chapter are reprinted from the *Builder's Journal*.

It may be taken that the foregoing schedule denotes what is required in a complete Court for an important assize town such as Cardiff, this schedule being taken from a paper read by Mr. H. V. Lanchester, A.R.I.B.A., before the Architectural Association in March 1905.

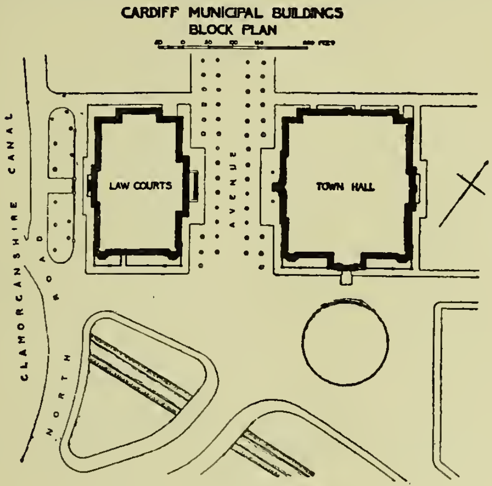
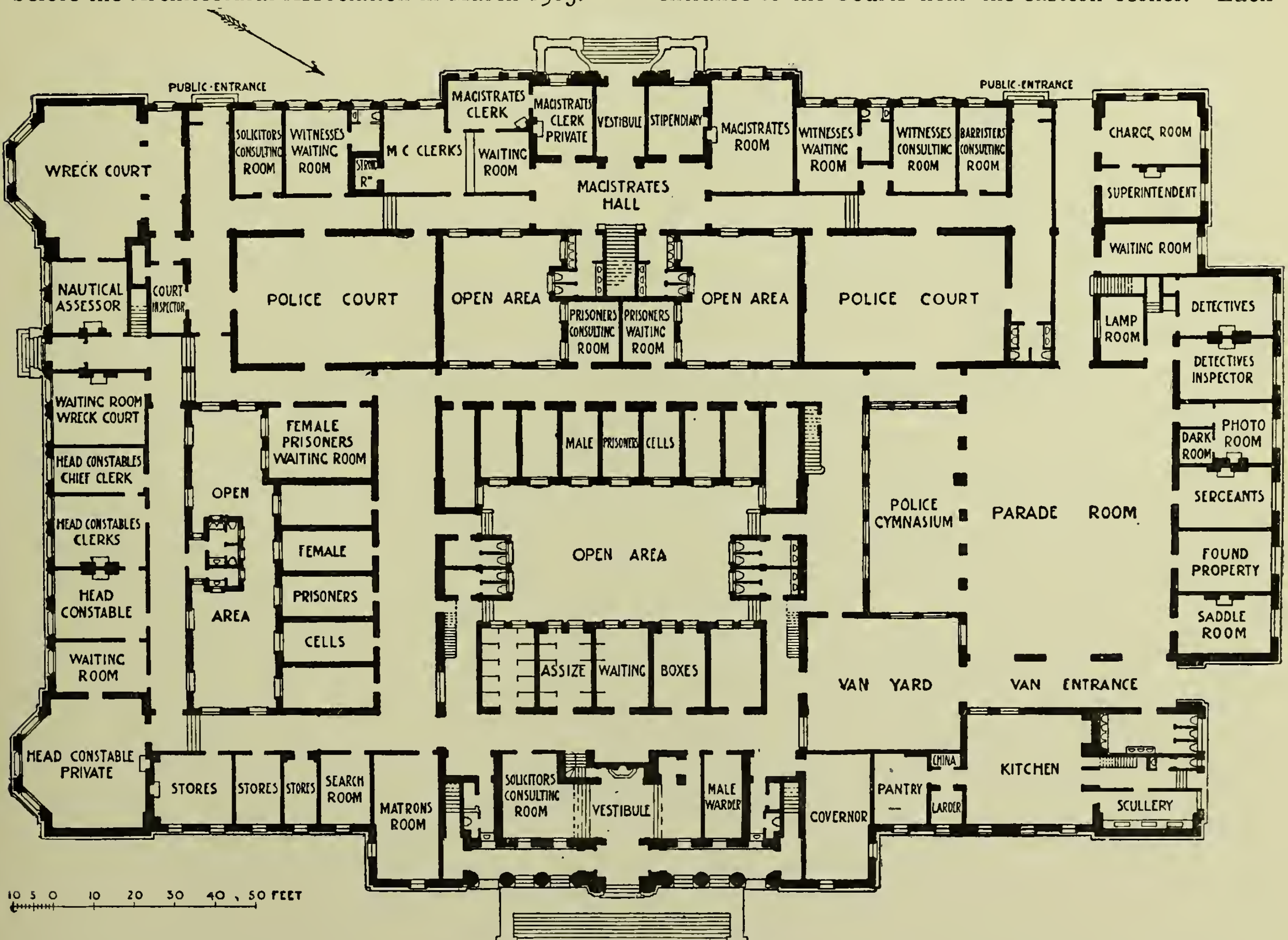


FIG. 16.

On reference to the ground floor plan (Fig. 17) it will be seen that the rectangular block has been made to enclose several open areas, and that the general scheme is based upon the necessity for placing the prisoners' cells and waiting-rooms so that there is no chance of escape to the open, these being in all cases lighted from the enclosed areas, and surrounded by other buildings. It then becomes necessary to provide separate access to all the Courts for prisoners, magistrates or judges, and witnesses, the internal communication being reserved for the prisoners, and external communication for the others. There is thus a magistrates' entrance, to be used also by the judges and jury of the Assize Courts, on the western side, with public entrances to the north and south of it near the extremity of the building. The general Assize Court entrance is on the east side, facing the Town Hall, while the police entrance is on the north, there being at Cardiff not only Courts but a complete police station in the same building. Dealing with this first, it will be noticed that it has only two entrances, one being a van entrance on the north side, and the other the ordinary police entrance, close to the public entrance to the Courts near the eastern corner. Each



GROUND PLAN.

FIG. 17.—Cardiff Law Courts.

Lanchester, Stewart, & Rickards, Architects.

is a long direct entrance, that on the east side passing, in sequence, the charge-room, the superintendent's room, and a waiting-room, straight through into a parade-room, the charge-room necessarily being the first reached, as that is the room into which prisoners are taken immediately on apprehension. After having the necessary entries made there, it is always essential that they can be passed straight to their cells, or, as in this case, to a large inner parade-room, whence escape is impossible. From the parade-room they can be taken down another long passage direct to the cells, of which those for male prisoners occur first and are shut off by a door from those for the female prisoners, these cells being quite close to the Police Court, where such prisoners would be tried almost immediately. In the event of conviction they would not be retained in these cells, but sent off to the gaol, and for this purpose they would be brought police court prisoners, who generally have to spend one and sometimes several nights at the police station awaiting the magisterial investigation. The other necessary parts of the police office comprise a gymnasium opening out of the parade-room, both these rooms being top lighted, and a series of rooms for detectives, photography, sergeants, found property, etc., all opening out from the parade-room, much as the rooms belonging to a railway station open out on to the platform. Over these rooms on the first floor (see Fig. 19) will be found the superintendent's residence on the north-west corner, and a general residence for the men, comprising kitchen and scullery together with a large men's room, reading-room and clothes store; a special drying-room being provided much like that in a laundry for drying wet clothing.

Reference to the plan (Fig. 17) will show that the corridors leading from the parade-room to the male

PLAN OF POLICE COURT CARDIFF

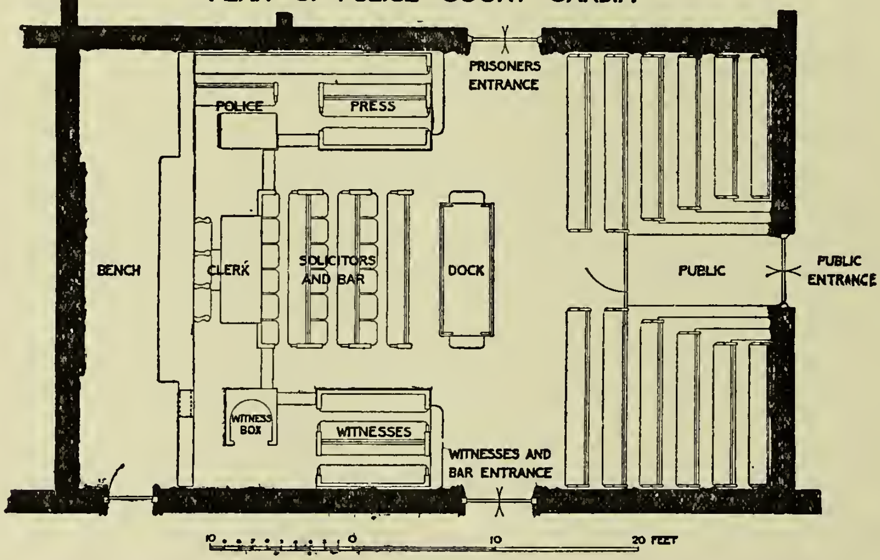


FIG. 18.

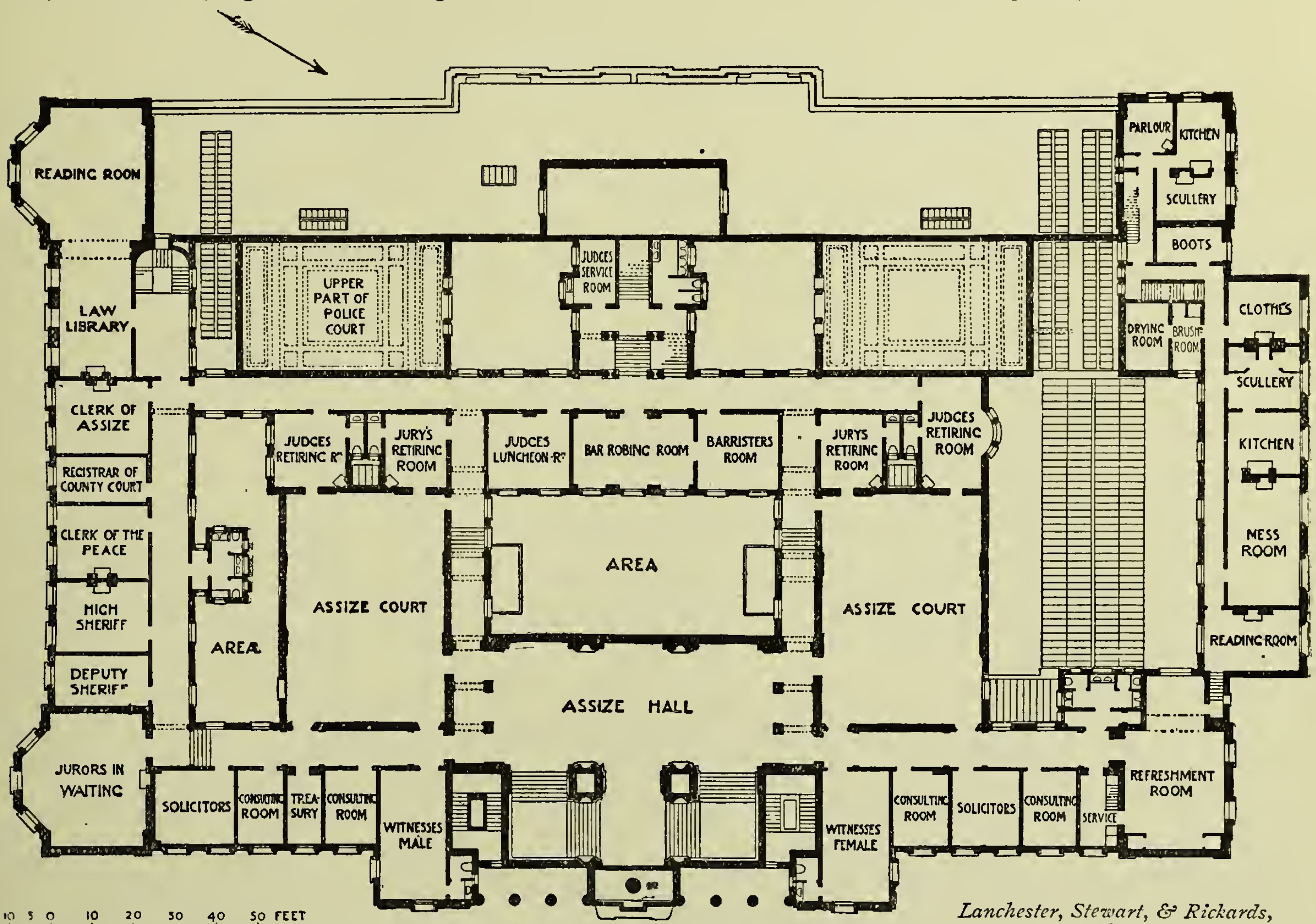
back through the corridors to the enclosed van-yard, placed in the vans and taken out through the van entrance, which crosses one end of the parade-room, from which it would be always in view. In this way, while a van is loading the outer gates would be closed, and they would only be opened when the van was perfectly ready for admission to the streets with its guards in their position. This van entrance and van-yard also serve a series of assize waiting-boxes, somewhat like cubicles or the boxes in the police van, arranged in groups in large enclosed cells, and intended for prisoners who had already passed through the ordeal of the Police Court and were now brought up for trial before the judge. Such prisoners spend only a short time in the boxes, as they are brought to them day by day from the gaols, and in case of a remand are taken back at night; consequently small boxes suffice for them, although large cells have to be provided for the prisoners' cells, and from the van-yard to the Assize Court boxes, are continued right round the building, forming internal means of communication to the various rooms. On the south-east this corridor leads to a series of rooms for the use of the head constable and his clerks, and at all points of termination of the corridor there are doors, which can be locked if needed to prevent all possibility of the escape of prisoners. By the use of a little care, internal lighting from areas is obtained to the corridor, rooms being interposed between it and the outer walls.

At the extreme south corner of the site there is a special small Court for nautical cases, known as a Wreck Court, with a direct external entrance for the public, and a room at one end of it for the nautical assessor. Along the south-west frontage are arranged a series of rooms for the use of the magistrates and their clerks, and for witnesses, solicitors, and barristers

engaged in the Police Courts. It is in the centre of this front that the magistrates' and judges' entrance is placed with a staircase immediately opposite to it leading up to the first floor.

Separating the rooms thus arranged along the frontage from the Police Courts there is a long corridor, so that the Police Courts are themselves entirely surrounded, with corridors on three sides and an open area on the fourth, from which high-level light is obtained, as may be seen on reference to Fig. 19, while they are also top lighted. The magistrates' corridor

The detail plan of one of the Police Courts is given in Fig. 18, from which this arrangement may be perhaps better understood. The prisoners' dock is in the centre of the Court. This is the usual position, but it is often reached by a staircase from below, which in this case is impracticable, as the Court and the cells are on the same level. It is always necessary that the prisoner should be able to face the bench, from which he should be distant about 12 or 14 feet, with the light shining on his face. The witness also should be at about the same distance, or perhaps a little less from



Lanchester, Stewart, & Rickards, Architects.

FIRST FLOOR PLAN.

FIG. 19.—Cardiff Law Courts.

is on two levels, the centre portion being higher than the extremities. Thus the rooms used by the magistrates and their clerks are at the level of the bench of each of the Courts, while the solicitors' and barristers' rooms are level with the body of the Court, and the few steps which interpose serve as a sufficient barrier to prevent unnecessary cross traffic. The public enter at the end of the Court, and so do not conflict with the solicitors and witnesses, while the prisoners are brought in on the opposite side. It will thus be seen that four different classes of persons are admitted to the Court by different ways without necessarily crossing one another.

the magistrate, and so placed that his face is visible to the magistrate, the examining solicitor or barrister, and the prisoner, and near enough for his words to be audible to all. It will be seen that the witness-box is placed at one side of the bench, which is raised some 2 feet above the Court level, there being seats for the magistrates' clerks on another platform, whose level lies between that of the Court and that of the bench. The solicitors and barristers are placed directly opposite to the bench, between the magistrate and the dock, while witnesses waiting for their turn to be called are on one side, in close proximity to the witness-box, and the press and police are on the other. The general public

are accommodated entirely at the back, and very often the benches allotted to them have to be used to a certain extent by witnesses who are in waiting for cases other than that immediately under consideration.

Fig. 19 illustrates the general arrangement of the first floor, in which a good deal is made of a handsome Assize Hall, reached by the main staircase from the main north-eastern entrance. It is always important to have a large crush hall like this outside an Assize Court, as there is a good deal of traffic through it by barristers, solicitors, and witnesses. Out of this hall opens in either direction a corridor like the one below, which passes round the enclosed areas and serves a series of rooms which obtain their light externally. At the northern corner this corridor comes to an abrupt termination in a large refreshment-room, served from the kitchen on the ground floor (see Fig. 17), passing, on its way to this refreshment-room, the female witnesses' room and some consulting-rooms for one of the Courts. In the other extension of the corridor there is the male witnesses' room, and further consulting-rooms for the other Court; and then, along the south-east frontage, it serves a series of rooms for jurors in waiting, and the various clerks of the Courts, with a large reading-room at the south corner. Off the south-western portion of the corridor are the rooms for the judges and barristers.

These arrangements are controlled to a large extent by the requirements of the Assize Courts, of which there are two. The main difference between these Courts and the Police Courts consists in the necessity for providing for a jury, while the public are rarely admitted to the floor of the Court, but only to a gallery served by an entirely different staircase, which may be seen passing up on either side of the main stairs on Fig. 19, but having no communication with the first floor. It is necessary that both the judge and jury should have retiring-rooms, entered directly from the bench and at the bench level without passing through any of the corridors. The prisoners are usually brought to the dock up a special staircase from below, and the back of the court is given up to witnesses and jurors in waiting. The detailed plan of one of the Assize Courts, given in Fig. 20, will show how these requirements have been met at Cardiff, where the principal difficulty appears to have been in so placing the witness-box that persons occupying it can be seen as well as heard by judge, jury, and bar. It will be noticed that the jury have not so good a view of it as they would have were their seats transposed with those of the press; but, on the other hand, if this change were made they would not be able to hear so well, and jurors are occasionally backward in asking a witness to speak up when they cannot hear him. The dock is a much larger apartment than it is in the Police Court, being made to accommodate, if necessary, a considerable number of prisoners and warders at one time, while the barristers and solicitors are given separate seats, the solicitors being immediately in front of the clerk's table, and on the floor of the Court, while the barristers' seats are arranged on a rising gallery, with the dock behind them at an even higher level. A similar rising gallery is devoted to the witnesses and jurors in waiting at the back of the Court. The same care has been taken to prevent cross traffic in the corridor as was exercised in the floor below, each of the entrances being at the most convenient spot for its purpose.

A little difficulty may at first be found, when comparing this plan with the general plan given in Fig. 19, in determining how the dock is reached by the prisoners

PLAN OF ASSIZE COURT CARDIFF

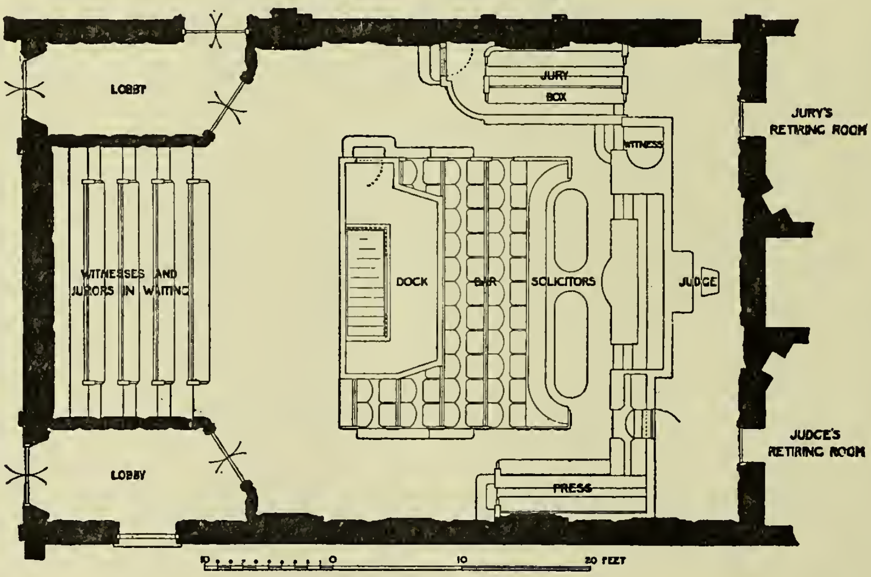


Fig. 20.

from the floor below, but a little consideration will show that the ground floor is sufficiently high to enable the stairs, shown on Fig. 17, on either side of the assize waiting-boxes, serving a passage way obtained over which it is reasonable should be tried in detail. The members are generally charged by the judge in one of the Assize Courts, but although two of these may be in use for trying cases by common juries only one

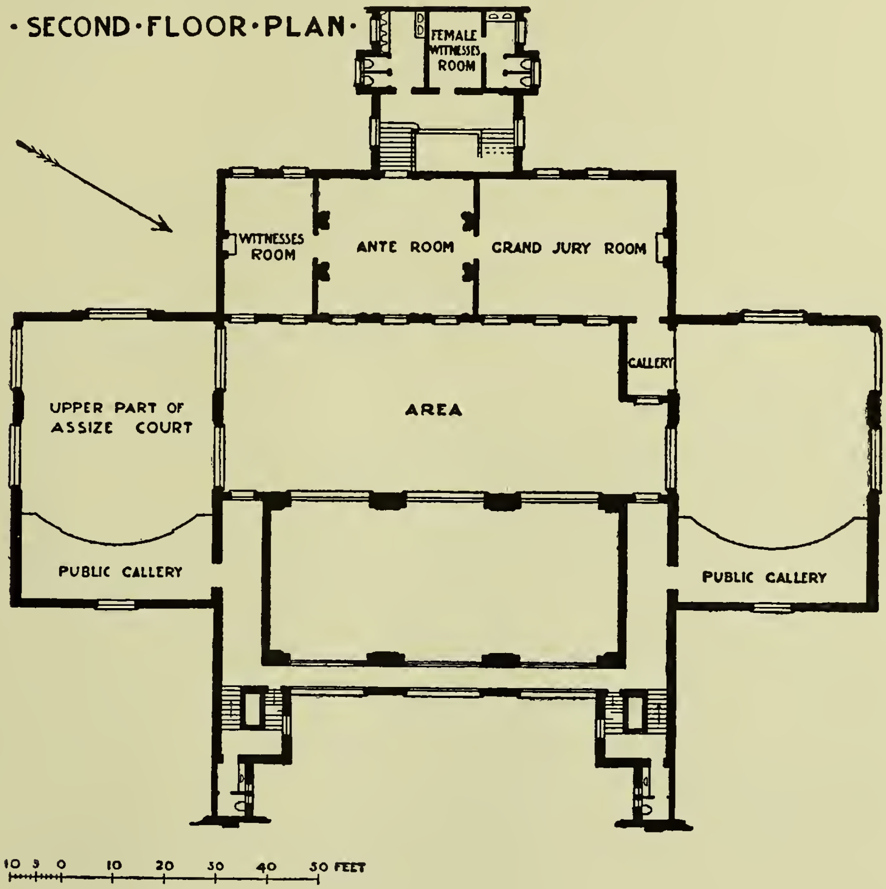


Fig. 21.

the ceiling of the prisoners' cells and below the floor of the Assize Court.

The second floor (Fig. 21), in addition to the public galleries of the Assize Court, contains only the grand jury room, with its ante-room and rooms for male and female witnesses. A grand jury consists of gentlemen drawn from the magistracy class, and is usually more numerous than a common jury. Its duty is merely to take sufficient evidence to assure that the case is one would be used for the grand jury. As each case is disposed of, the grand jury makes its report in a form known as a "Bill," which is handed to the judge of the Court in which the jury has been charged; and this can be done direct by means of a kind of fishing rod lowered from the gallery, shown attached to the grand jury room; the Bill in this way never passing into a corridor, and so running no risk of being tampered with.

CHAPTER V

LARGE MUNICIPAL BUILDINGS

The requirements of municipal buildings for larger towns are usually complex and varied to a large extent, with the result that it is difficult to lay down any definite rules for guidance, each problem having to be considered entirely by itself. Possibly the most useful thing to do under such circumstances is to describe the solution arrived at in certain well-known cases. It may be taken as a general rule that such buildings consist of the municipal offices proper, including the council chamber and several committee-rooms, and occasionally a series of rooms intended for reception purposes. Second in importance to these come the administrative departments, consisting of town clerk's office, and ranges of offices for the engineer and surveyor, the accountant, the rate collector, the auditor, and the medical officers, with special departments in certain boroughs for exceptional offices, such as those of the water-works manager. In very many cases a large town hall, intended for public meetings, concerts, etc., has to be provided

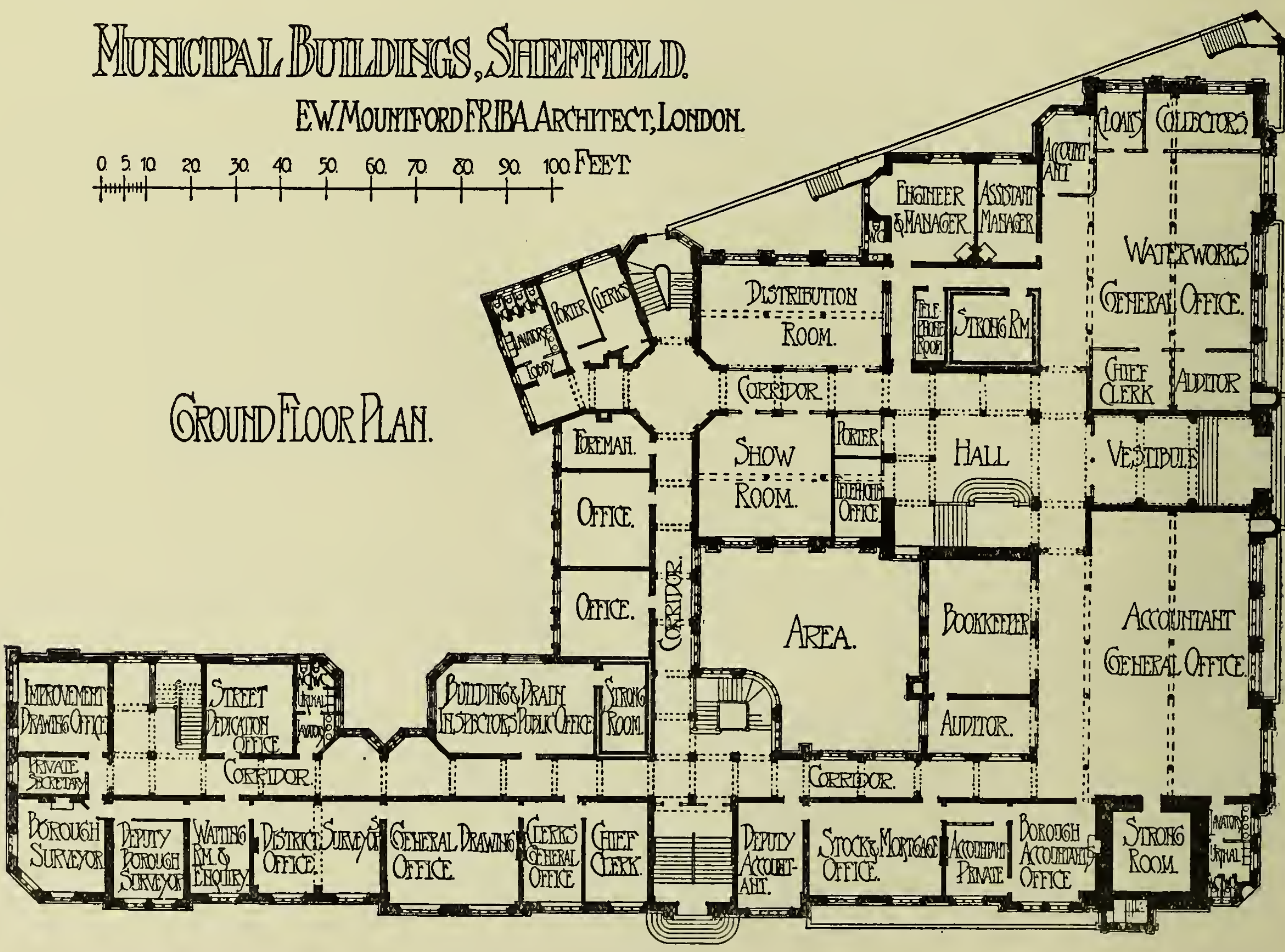


Fig. 22.

also; while Law Courts are not uncommonly included within the same building, and sometimes the fire brigade has also to be housed within it.

The municipal buildings at Sheffield belong to the more simple class, for neither town hall nor Law Courts had to be provided. The design carried out was that by Mr. E. W. Mountford, F.R.I.B.A., illustrated in Figs. 22, 23, and 24. It is perhaps most

convenient to consider the ground-floor plan first (Fig. 22), although obviously the first floor, which contains the council chamber and the reception-rooms, has to a large extent controlled the arrangement. It will be seen on reference to Fig. 22 that the general scheme is that of grouping the building round a central area, which is enclosed by a corridor off which the various rooms are opened; although the corridor does not in every case adjoin the area, and one arm of it is continued so as to serve the offices which are arranged along the side street. The main entrance is axial; for axial planning is almost essential when for communication between departments. Besides the main entrance there are two others on this floor from each of the side streets, which have to be reached in both cases by means of stairs. These two are at the extremities of one of the arms of the corridor, the various corners of which are marked by large lobbies, and in one case by a secondary staircase. Along this arm of the corridor there is direct communication from one side through to the other, and in this way another department, that of the borough surveyor, is cut off, and to a great extent rendered self-contained, though it consists of a number of small offices which

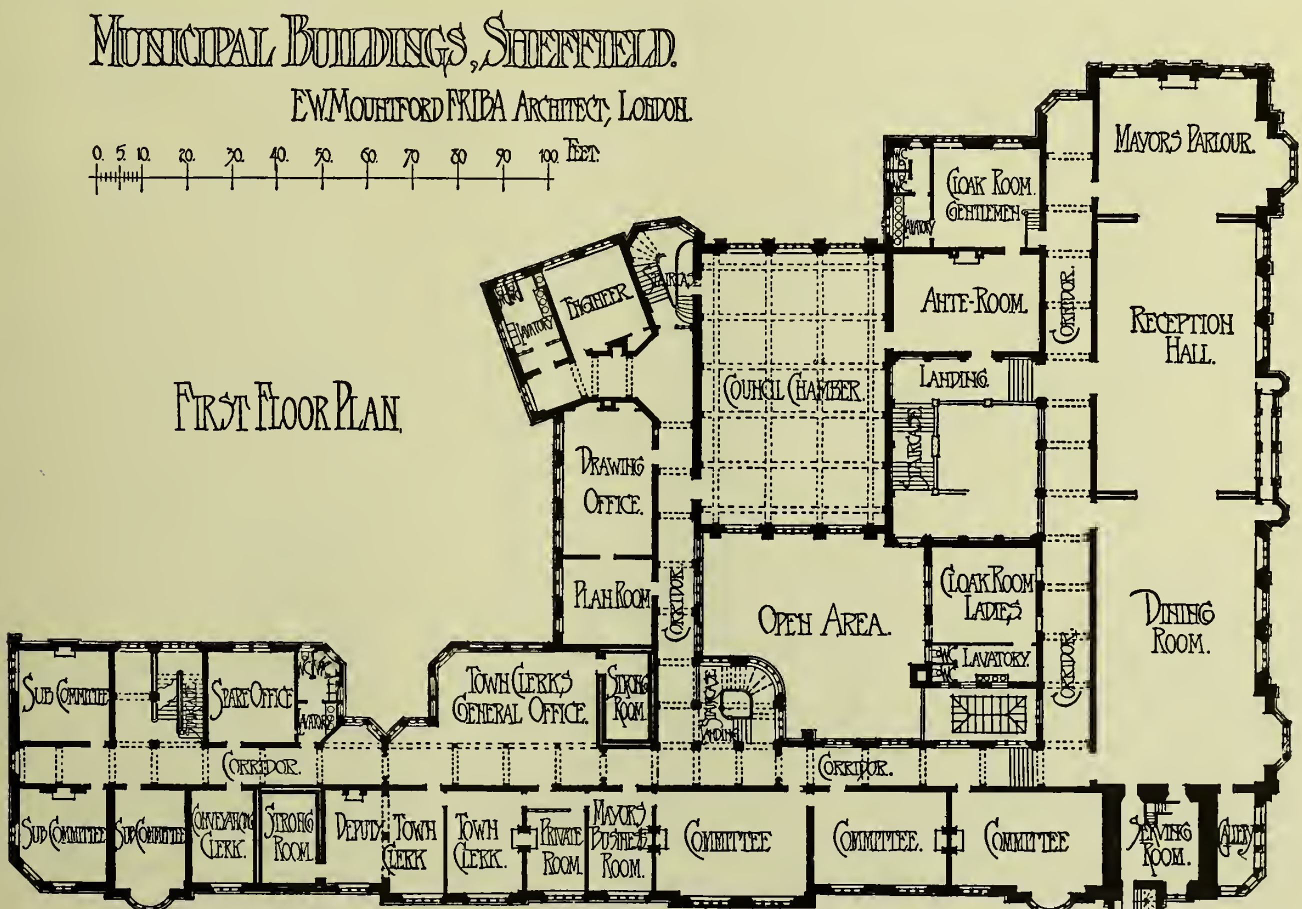


FIG. 23.

dealing with big buildings. A broad vestibule leads to a large staircase hall traversed by the corridor already mentioned, which, however, though apparently continuous on plan, is really cut into sections by doors at either end of the accountant's office, which is entered on the left hand of the hall. This department is self-contained, all its rooms being easily served from the general office, and being in many cases intercommunicating or supervised through glazed partitions. Corresponding to this on the other side of the vestibule are the offices of the water-works manager, similarly self-contained and with perfect intercommunication between its various rooms, the corridor being utilised open out of one another, and not of a great central office from which minor offices are reached. A third staircase at the end of this wing is also provided.

If we turn to the first-floor plan (Fig. 23) we find that the main staircase has a comparatively low-level landing from which the ante-room to the council chamber is reached, this being top lighted and having a cloakroom out of it on one side, while it gives access to the chamber by a narrow door at the other end. The chamber is a long rectangular apartment with windows at either end, sufficiently recessed from the side road for the traffic to be comparatively inaudible. It has two other doors, one on to a staircase which

passes down to the entrance by one of the side streets and so serves for the public, while the other is intended for the use of the executive officers, as it is approached readily from the engineer's and town clerk's offices on this floor, and by means of a staircase from the surveyor's and accountant's offices on the floor below.

The whole of the front of the building is given up to a series of reception-rooms at a somewhat higher level than the council chamber and the offices. These consist of a mayor's parlour, a reception hall, and a dining-room, arranged *en suite*, the main entrance to the reception hall being immediately opposite the main staircase landing. Service to the dining-room is well arranged, with a small staircase and lift in an external tower, passing up to the kitchen and caretaker's rooms on the second floor, and down to a side entrance and the coal-stores in the basement. The committee-rooms are arranged along the side street next to the serving-room, and are more conveniently approached from the town clerk's office than from the council chamber, in connection with which they are not often needed. Immediately communicating with the town clerk's private room is the mayor's business room. These again are on the lower level, and the few steps necessary for marking the difference between one level and another serve admirably to distinguish between the purely ornamental portions of the building, comprised in the reception-rooms, and those parts which are devoted to business.

The basement (see Fig. 24) necessarily reflects the floors above. Along the front it is given up to storage, while a large heating chamber is arranged in the middle. The main staircase is not carried down, but the space beneath it forms a large vaulted hall. There

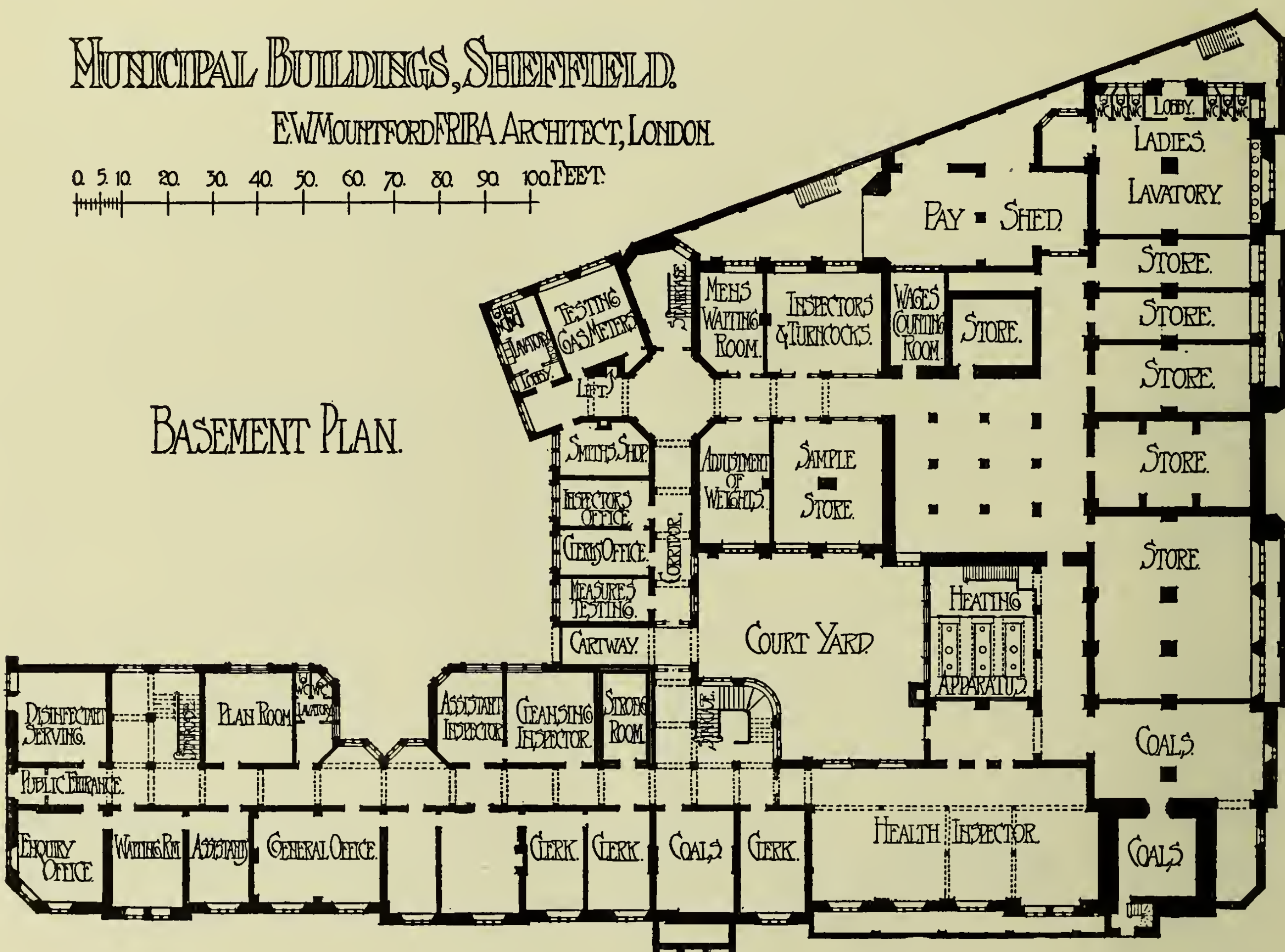


Fig. 24.

is a wages counting-room, with a pay shed cleverly arranged close to it, so that the outdoor employees can wait under cover while they are paid and pass in a regular stream in front of the pay counter. A series of offices is arranged on this floor for heavy purposes, such as the adjustment of weights and testing gas meters, while a cart-way admits to the courtyard in the central area. There is one public entrance at this level, at the extremity of the long arm of the corridor, and this serves to admit to such offices as are placed near to it.

Another similar building for a large provincial

municipality is the Walsall Town Hall, which has recently been completed from the plans of Mr. J. S. Gibson, F.R.I.B.A., the principal additional requirement in this instance being that a large assembly hall should be provided for public functions of all sorts, and by this means the hall can be used without interfering with the general work of the building. It had necessarily, in addition to this, to be accessible both from what may be called the reception portion and the office portion of the municipal buildings. How

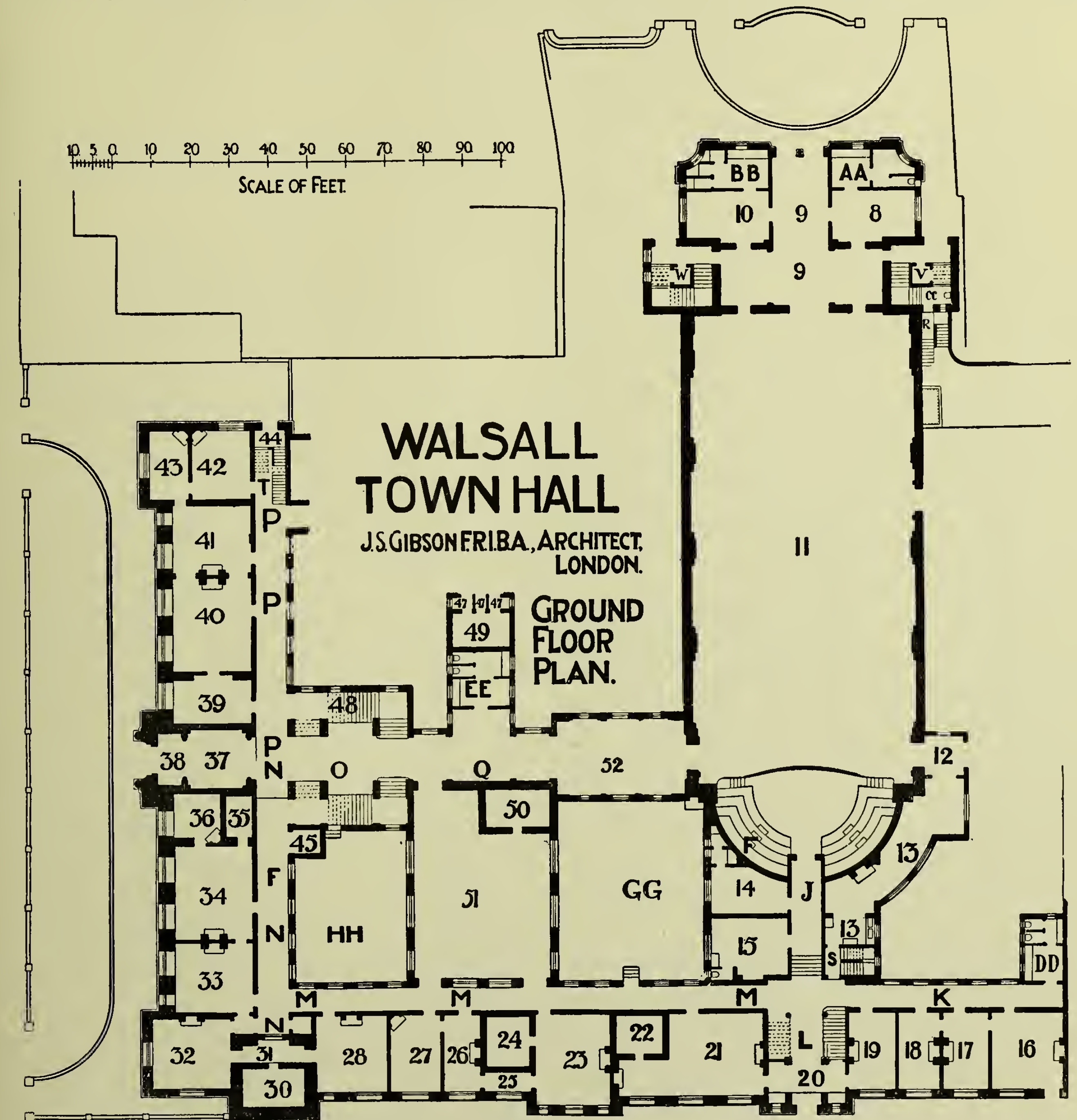


Fig. 25.

including concerts. The site was such that two main frontages were obtainable at right angles to one another, while there was a narrow frontage to another important street in the rear. This narrow frontage was utilised for the great hall, while the main frontages supplied the entrances to the municipal offices proper; cleverly this has been done will be seen by reference to the ground plan (Fig. 25), with which the following list of rooms must be compared—as the plan, owing to the small scale of its reproduction and its great extent, has had the rooms upon it indicated by numbers and reference letters only.

WALSALL MUNICIPAL BUILDINGS.

List of Rooms.

Basement.

1. Heating Chamber.
2. Chair Store.
3. Chair Store.
4. Chair Store.
5. Service Room.
6. Office Coal Store.
7. Office Coal Store.

Ground Floor.

8. Gents' Cloak Room.
9. Entrance Hall to Town Hall.
10. Ladies' Cloak Room.
11. Town Hall.
12. Exit.
13. Kitchen.
14. Ladies' Retiring Room.
15. Gents' Retiring Room.
16. Weights and Measures Inspector.
17. Weights and Measures Waiting Room.
18. Adjusting Room, Weights and Measures.
19. Health Department—Clerks' Office.
20. Entrance Hall from New Street.
21. Justice Clerk's Office.
22. Justice Clerk's Strong Room.
23. Borough Accountant's Clerks' Office.
24. Borough Accountant's Strong Room.
25. Borough Accountant's Corridor.
26. Borough Accountant's Waiting Room.
27. Borough Accountant's Private Room.
28. Town Clerk's Office
29. Town Clerk's Cupboard.
30. Town Clerk's Strong Room.
31. Town Clerk's Strong Room.
32. Town Clerk's Private Room.
33. Town Clerk's Office.
34. Town Clerk's General Office.
35. Town Clerk's Strong Room.
36. Town Clerk's Typewriter.
37. Principal Entrance Hall.
38. Principal Entrance Vestibule.
39. Gas Department—Stock Room.
40. Gas Department—Exhibition Room.
41. Gas Department—Gas Cooker and Stove Room.
42. Gas Department—Meter Inspector.
43. Gas Department—Testing Room.
44. Entrance.
45. Lift.
46. Porter.
47. Official Lavatories.
48. Telephone Exchange.
49. Official Lavatories.
50. Borough Accountant's Strong Room.
51. Borough Accountant's Collectors' Office.
52. Crush Hall.

First Floor.

53. Refreshment Room.
54. Store.
55. Servery.
56. Dining Room.
57. Still Room.
58. Store.
59. Health Department—Inspectors' Office.
60. Health Department—Chief Inspector.
61. Health Department—Clerk's Office.
62. Health Department—Medical Officer.
63. Health Department—Disinfectant Store.
64. Spare Room.
65. Borough Engineer—Foreman's Office.
66. Borough Engineer—Building Inspector.
67. Borough Engineer—General Office.
68. Borough Engineer—Strong Room.
69. Borough Engineer—Private.
70. Borough Engineer—Assistant Surveyor.
71. Overseers' Department—Office.
72. Overseers' Department—Office.
73. Town Clerk's Strong Room.
74. Overseers' Department—Strong Room.
75. Gas Department—Manager's Room.
76. Gas Department—Waiting Room.
77. Gas Department—Clerks' Room.
78. Gas Department—Strong Room.
79. Gas Department—Passage.
80. Gas Department—Ledger Clerk.
81. Cloak Room.
82. Ante-room.
83. Council Chamber.

Second Floor.

84. Store.
85. Service Room.
86. Kitchen.
87. Scullery.
88. Larder.
89. Spare Room.
90. Spare Room.
91. Electrical Engineer's Office.
92. Electrical Engineer's Office.
93. Gas Department—Store.
94. Gas Department—Store.
95. Borough Engineer's Testing Room.
96. Borough Engineer's Spare Room.
97. Borough Engineer's Drawing Room.
98. Borough Engineers Strong Room.
99. Library.
100. Retiring Room.
101. Committee Room.
102. Committee Room.
103. Committee Room.
104. Ante-room.
105. Mayor's Parlour.

Attic Floor.

106. Service Room.
107. Store.
108. Kitchen.
109. Bedroom.
110. Bedroom.
111. Cupboard.
112. Bedroom.
113. Sitting Room.
114. Larder.
115. Scullery.
116. Bath Room.
117. W.C.
118. Store.
119. Store.

The public entrance from the back street is by way of a vestibule and crush hall (No. 9) with cloakrooms 8 and 10 on either side, to the large hall (No. 11), the galleries being served by the two staircases V and W, to which there is an entrance and exit as well as communication from the crush hall. A semicircular platform occupies the other end of the hall, but just before it is reached there are emergency exits on either side, through lobby No. 12 into an open yard on one side, and through another crush hall (No. 52) to the reception entrance

No. 38. By means of No. 12, through communication with a kitchen (No. 13), which is inserted here for the purpose of serving banquets in the great hall, is obtained, and through this kitchen the corridor M of the office portion of the building is reached, as is also the office entrance No. 20. From corridor M a few steps lead down to the performers' entrance J at the back of the stage, from which are obtained two artistes' cloakrooms (Nos. 14 and 15).

The large crush room (No. 52) serves as the entrance when the great hall is used for municipal purposes, such as banquets, just as the crush hall No. 9 serves when it is being used by the general public or for entertainments. This (No. 52) is approached through the main municipal entrance (Nos. 38 and 37), and by way of the large staircase hall O, making an even finer approach than is that for the general public.

It is a very noticeable feature in this admirable plan that all three main entrances—for the public, for municipal officers, and for the performers—are absolutely direct from the outer door to the hall.

Like the Sheffield Town Hall, the building is served by a corridor which gives internal means of communication to a number of rooms ranged along the two main streets. The reception entrance (No. 38) is in the centre of one of these frontages, and leads directly into the great staircase hall, on one side of which (No. 45) is a lift communicating with all the floors, and on the other side (No. 48) is a telephone exchange, a most useful feature not very often found.

As in all similar buildings, each department is so arranged as to be self-contained and inter-communicating, while communication with other departments is managed by means of the corridors, and of course by telephone. Along the arm of the corridor P is arranged the gas department rooms (Nos. 39, 40, 41, 43, and 42), which are *en suite*, the two last being for the meter inspector and for testing; while No. 40 is a large room to be used for gas exhibits. A similarly intercommunicating suite of rooms (Nos. 28 to 36), are for the use of the town clerk, whose private office (No. 32) is at the corner of the building, so arranged as to be equally well accessible through the public office (No. 33), or directly from the corridor. The private strong room passage (No. 31), which gives communication between the town clerk in No. 32 and his chief assistant in No. 28, will be noticeable. The group of rooms Nos. 23 to 27 are for the use of the borough accountant, who, while he has a good deal to do with the town clerk, is not often approached by the public, and so is not placed very close to either of the entrances, though he is in exceedingly handy proximity to the large collector's office No. 51, which is entered from either end. This room, having two entrances into corridor M, will permit of a stream of people passing in at one door and out of another in case of necessity. Room No. 21 is devoted to the justices' clerk, and can be approached immediately from entrance No. 20, as also can the clerks' office of the health department (No. 19); these both being rooms which the public often need to enter. Rooms 16, 17, and 18, placed at the extremity of corridor K, belong to the weights and measures department, and of these No. 16 has an external entrance.

A little consideration of the plan will show that it has been based upon axes drawn with considerable care, one of them extending from entrance 9 to entrance 20 right through the centre of the large hall, and the other drawn from the centre of the other main frontage, down entrance 38 to crush room 52; and that beyond this the plan is a courtyard plan, the great courtyard being cut up into three parts, forming the areas HH and GG, separated from one another by room 51; and it is round this large courtyard that the corridors are worked.

The first-floor plan (Fig. 26) perhaps exhibits the general scheme more clearly than does that of the ground floor, which to a large extent it has controlled, as frequently happens in the case of large buildings where important rooms are placed on an upper floor. The large hall (No. 11) is now isolated, the space at the back of the platform being given up to an organ, and a large room (No. 53) over its entrance being arranged for refreshments, at the level of the central portion of the gallery, and served by the gallery stairs. The first floor is thus little controlled by the town hall, while on the other hand its principal feature is the council chamber (No. 83), which is placed in the middle of the site in the most quiet spot possible, and lighted from both sides. The means of access to it is somewhat unusual. The main staircase, after reaching at O the level of the corridors P, N, and M, is continued by a flight of eight steps, shown perhaps more clearly in Fig. 26A, to the level of the ante-room No. 82, from which the council chamber is entered by two doors, the cloakroom (No. 81) being on the opposite side of the ante-room. There is no entrance at this level from the farther end of the room, at which there is a gallery arranged for the public. By this curious arrangement of staircases and landings the council is entirely cut off from the mere official parts of the building, served by the corridor and comprising on this floor all rooms from Nos. 54 to 80. Those numbered 75 to 80 are reached by the main staircase, and comprise the gas department, being *en suite*, as in all such cases, with the strong room carefully isolated and enclosed. No. 73 is an additional strong-room for the town clerk, with its door immediately opposite the few rising stairs to the ante-room of the council chamber, it being placed here and not among the town clerk's offices, for the storage of papers which are likely to be required at council and committee meetings. Rooms 71, 72, and 74 for the overseers' department,—No. 74 being the strong-room. The borough engineer has one of the largest suites of rooms in the building (Nos. 65 to 70), the private offices for the engineer and his chief assistant being Nos. 69 and 70,

in the corner of the building, whence access to and control of the staff in No. 67, the long general office, is easy. The drawing-office (No. 97), together with a testing-room (No. 95) and a spare room (96) will be found on the second floor (Fig. 27), immediately over the staircase L, and so can easily be reached from the street. A series of small rooms (Nos. 54 to 58) act as staff dining and refreshment rooms, and are served from the kitchen above (No. 86) by means of a special staircase S, and a lift which runs up all three storeys,

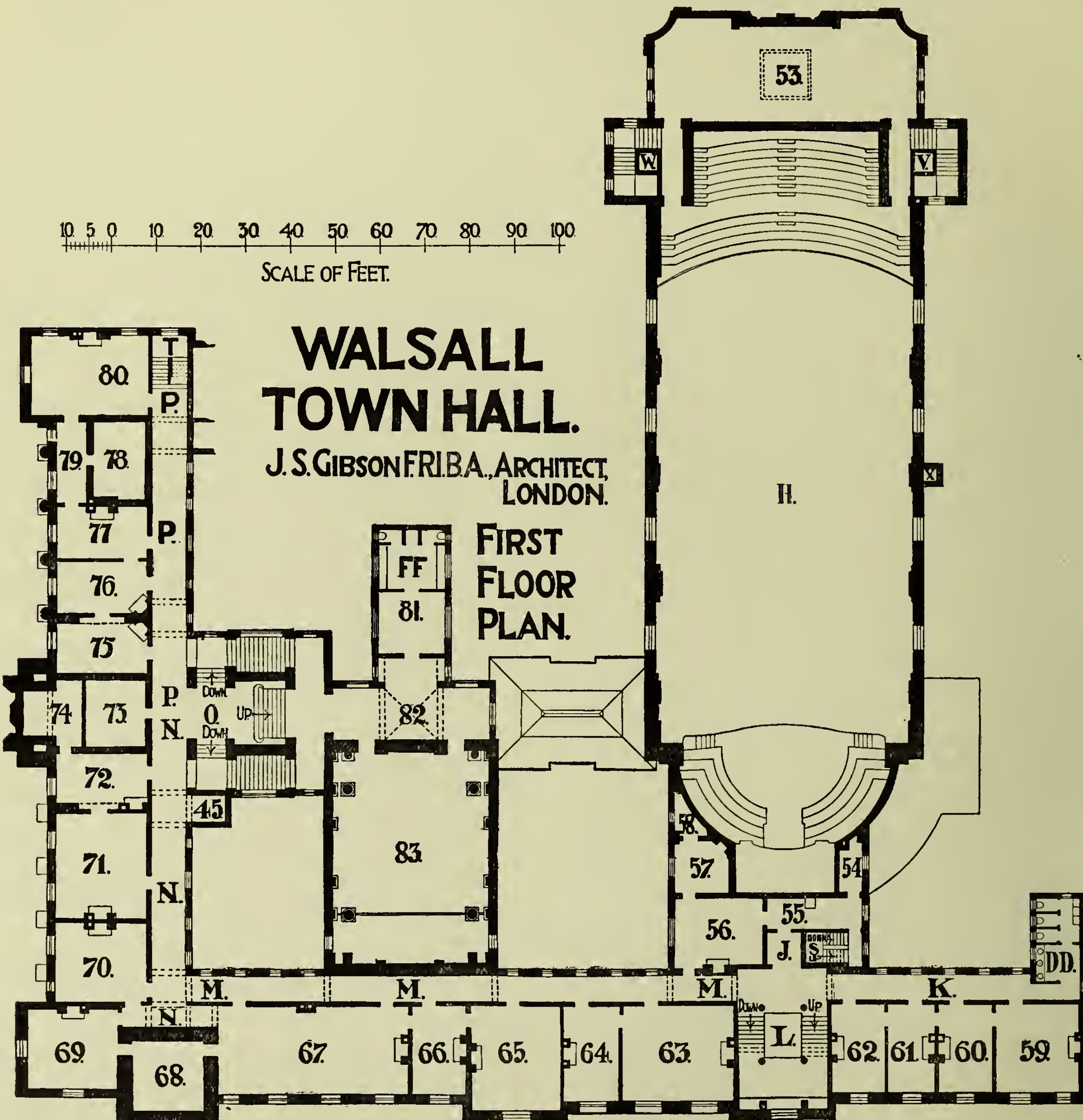


Fig. 26.

these offices, but not very easily reached, as there is no direct staircase—though communication by speaking tube would of course be a simple matter. The health department is located on the first floor, in rooms 59 to 63, the last of these being detached from the others to form a store for disinfectants. These are all close to shown in No. 85 on the second floor and No. 13 on the ground floor.

The same general arrangement is followed on the second floor, the plan of which (Fig. 27) needs reading with a little care, as although the hall (No. 11), and the cloakroom, ante-room, and council chamber (Nos. 81,

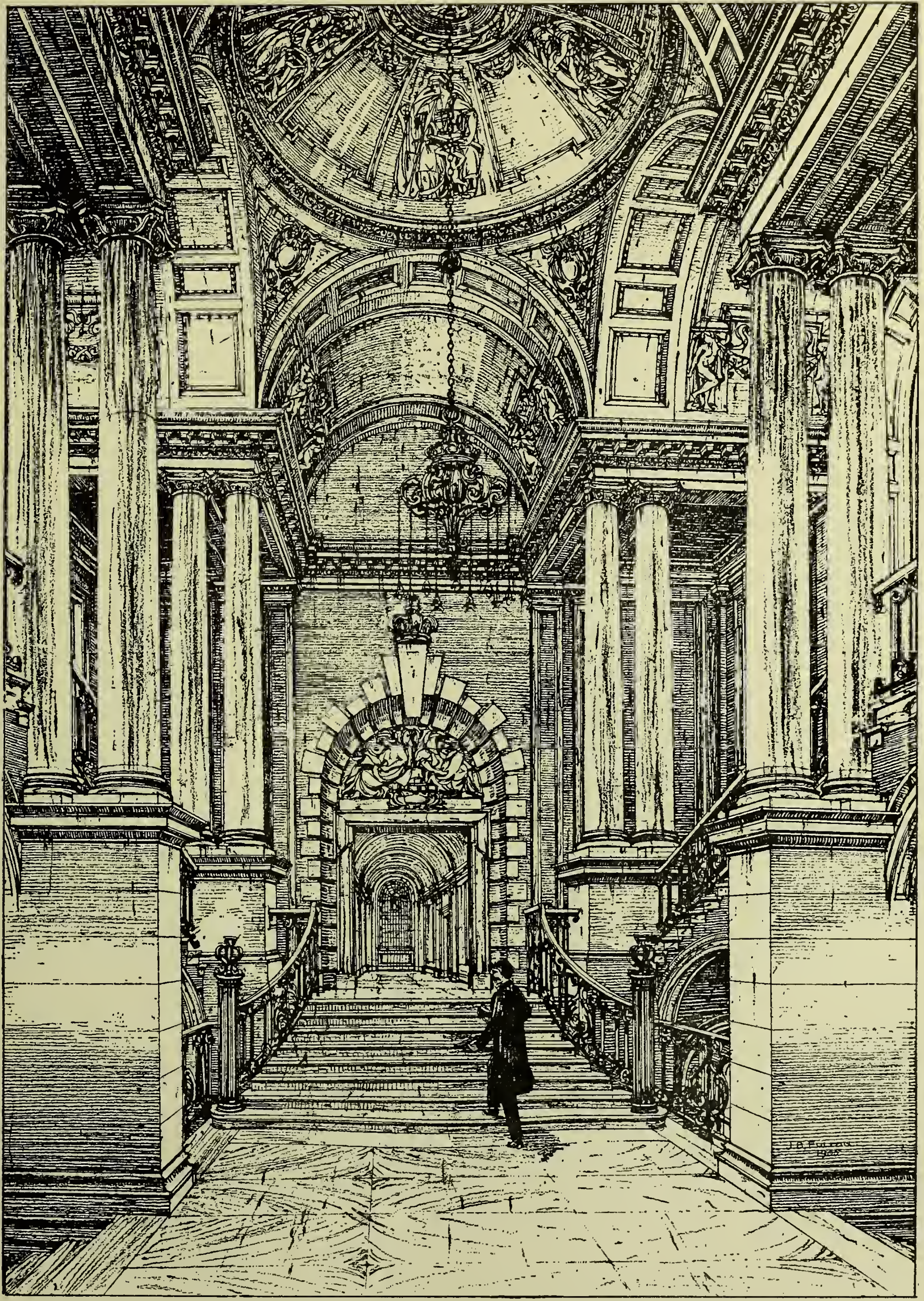


FIG. 26A.—Walsall Town Hall (View across Staircase). [*J. S. Gibson, F.R.I.B.A., Architect.*

82, and 83) are all shown thereon, these really occur, as already explained, at the half-level between the two floors, and at this ground floor level it is only the gallery of the council chamber (No. 83) which is reached by the public off corridor M. Rooms 84 to 88 form a range of kitchens for serving the staff dining-room below. A small electrical engineer's department is located in Nos. 91 and 92, while a couple of spare rooms occur in Nos. 89 and 90 at the end of corridor K. Nos. 93 and 94 are used as stores for the gas department.

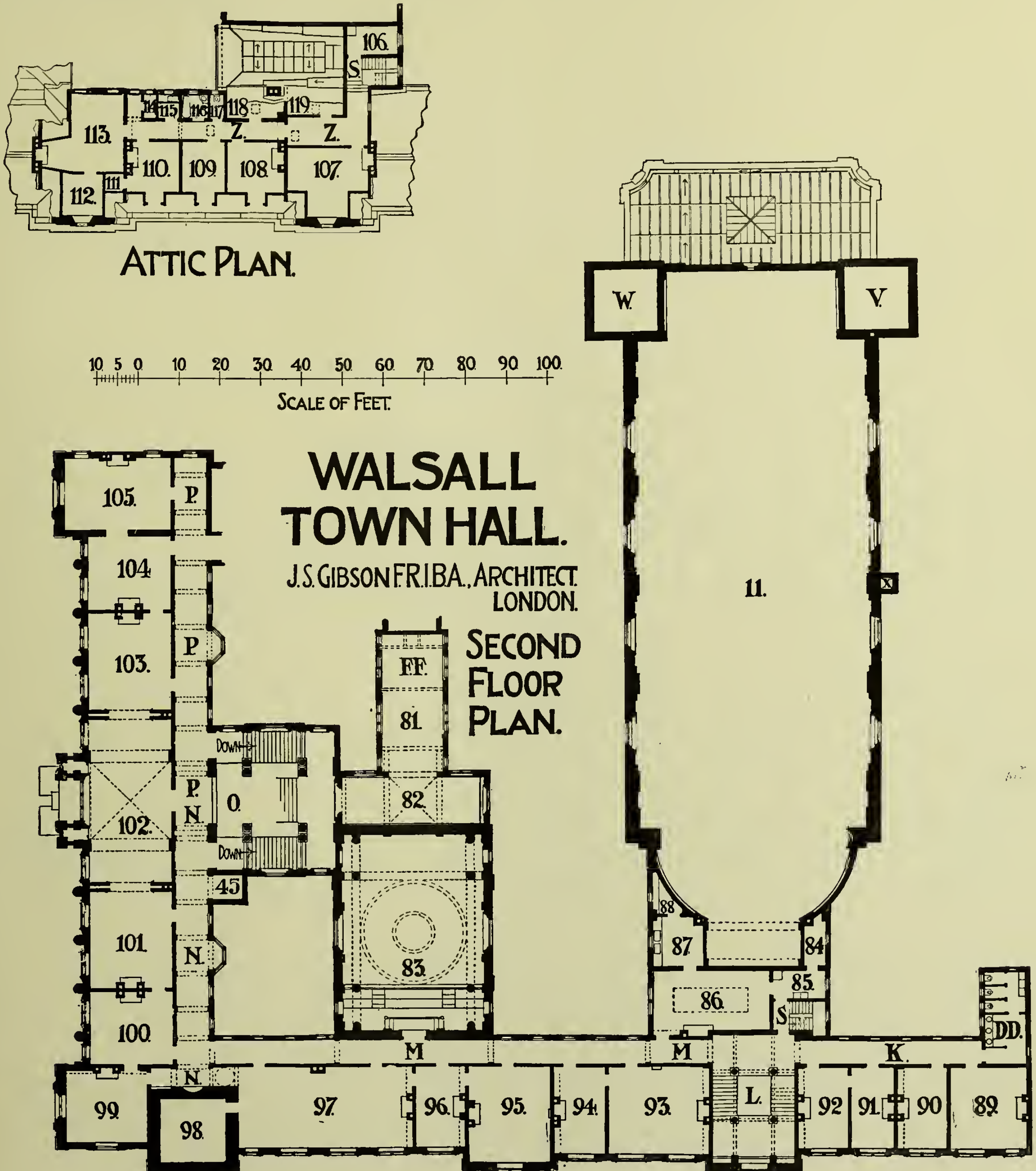


FIG. 27.

Nos. 95 to 98, as already said, form a portion of the engineer's department, and would house the drawing staff, while the strong-room No. 98 would be used almost entirely for the storage of plans. The front rooms

on this floor, from Nos. 99 to 105, are all committee-rooms and reception-rooms for the use of the council, being served by the main staircase O. These are all *en suite*, so as to be used when needed for receptions.

A small attic floor, the plan of which is also shown on Fig. 27, is devoted to a caretaker's residence. It is reached by the service staircase S, and extends over the secondary staircase L, and along part of the frontage served by that staircase, including its two gables.

The Lambeth Municipal Offices very nearly resemble those of Walsall in their requirements, with the exception that the town hall is to be built subsequently and to be detached. Considering how satisfactory is the result at Walsall of planning for communication, it is a pity that the plans at any rate were not prepared at the outset for both buildings, even if the municipal offices were to be first built. However, the conditions of the competition precluded this, and the successful design was that of Messrs. Warwick & Hall. As shown in Fig. 28, the site was an extremely awkward one, coming to a sharp angle at the junction of Brixton Hill and Acre Lane, it being necessary to provide architecturally satisfactory frontages to both of these important thoroughfares. Messrs. Warwick & Hall put their principal entrance at the corner, reached by an external flight of steps, and based their plan on an axis obtained by bisecting the sharp angle, splitting up the corridor into two radial lines from an inner hall, so as to serve a series of offices along either frontage.

There is a large circular vestibule entrance, with rooms for the porter and telephone on either side, and well lighted in front. A flight of seven stairs leads thence to the inner hall, and on its way passes the main staircase, which rises both to right and left clear of the entrance corridor. The arrangement of this portion of the plan is unusual, and worthy of some considerable amount of study. The two radiating corridors are connected across what may be termed the base of the triangle by a large rates office, served by entrances on both roads, so that persons can pass in at one entrance, pay their rates, and go out into the other street. The Brixton Hill entrance bisects its frontage, and so forms an axis from which it can be designed. The general departmenting, to which attention has been drawn in several other cases, will be noticeable on this plan. All the borough engineer's offices lie off the Acre Lane branch of the corridor, while those for the borough accountant are in the same way ranged along the Brixton Hill frontage, between the main entrance and the secondary entrance on that side, the remainder of that corridor giving access to the medical officers' quarters, to which there is a special entrance at the extreme end of the corridor. The borough accountant is also placed in close proximity to the rates office, from which a small staircase leads to the basement floor (see Fig. 29), where it serves a series of rooms devoted to rates appeals, which, like the rates office above, can be reached from both the Brixton Hill and the Acre Lane entrances.

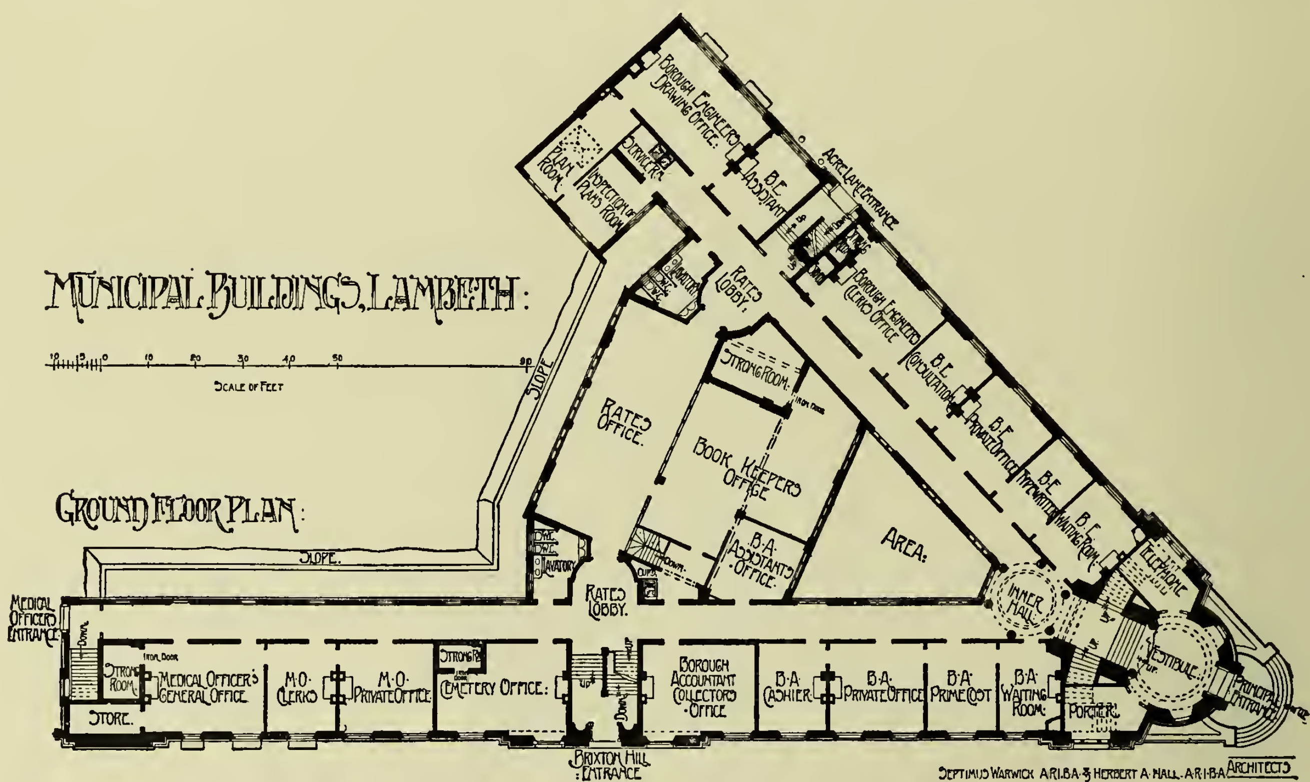


FIG. 28.

The staircases from these entrances serve this floor, as the main staircase is not carried down, and most of the rooms obtained are storerooms, though an external corridor within an open area will be noticeable leading from side to side in front of the windows of the rates appeals room, and giving access

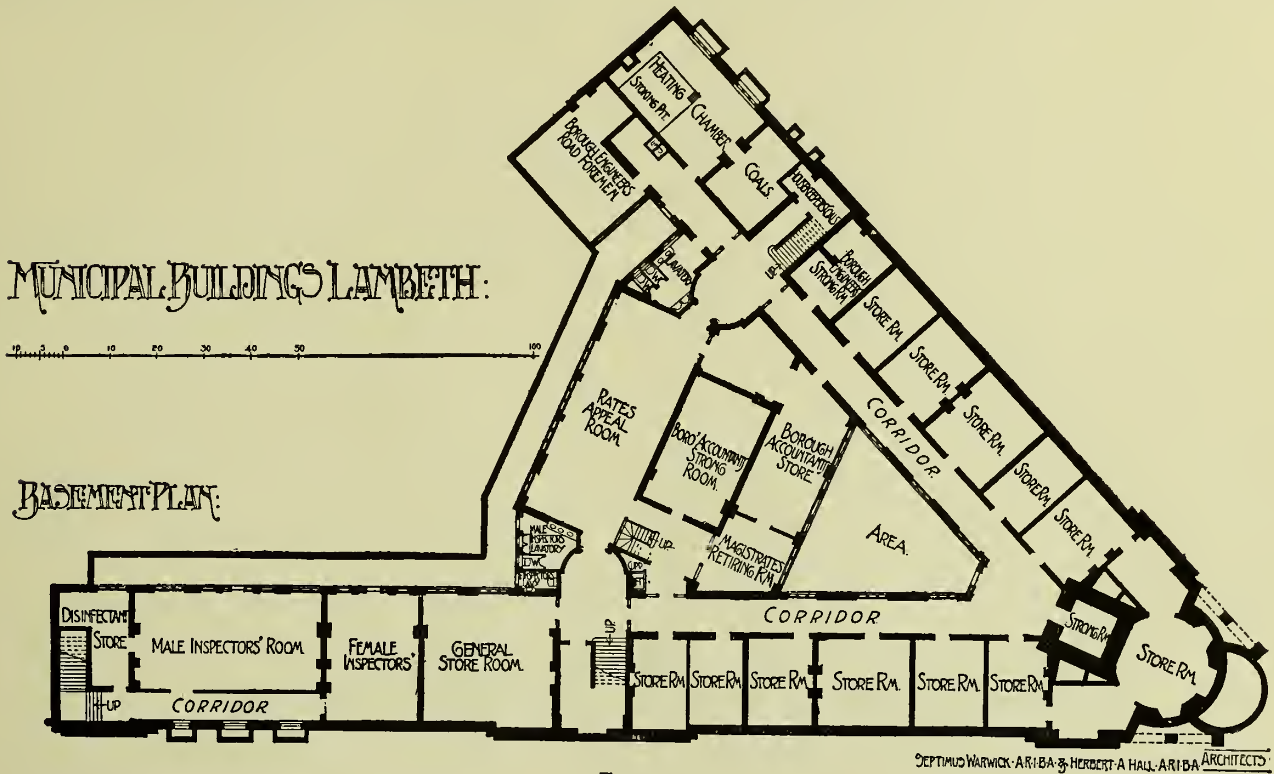


FIG. 29.

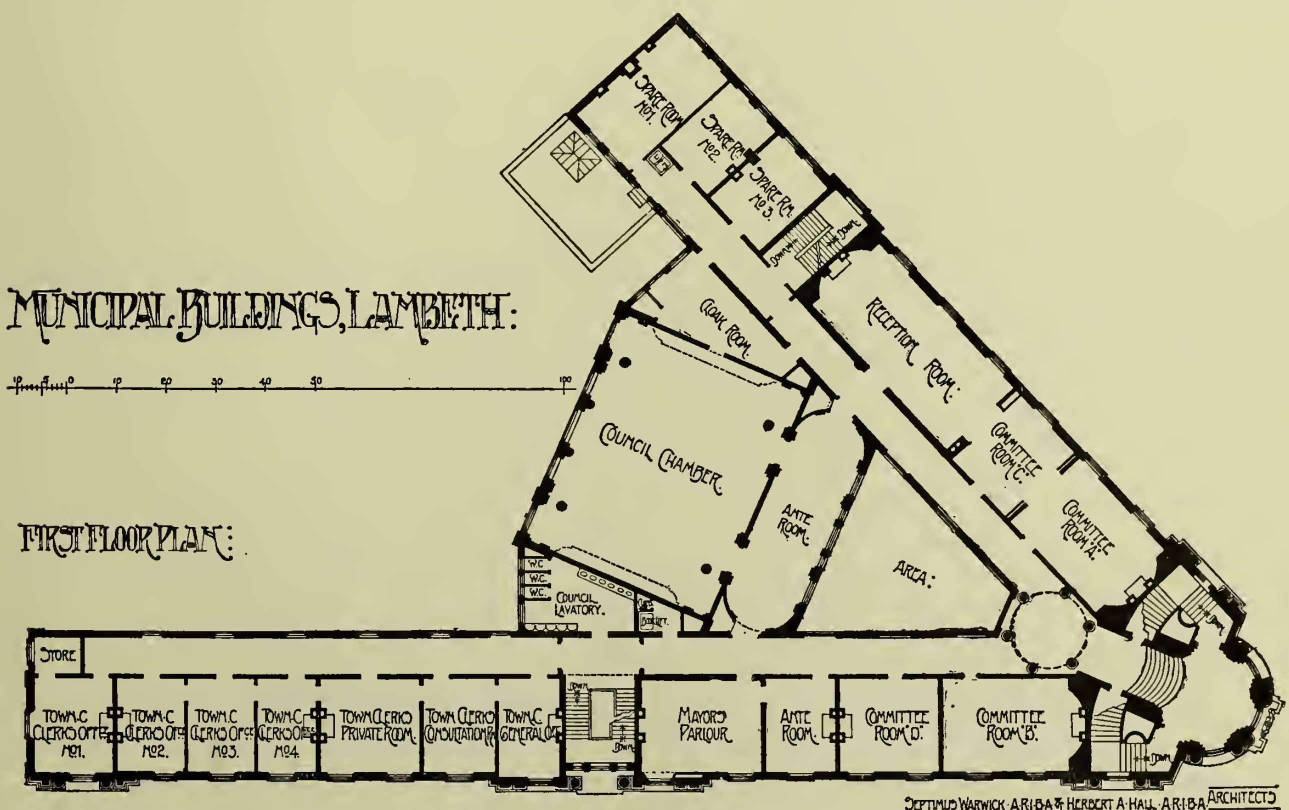


FIG. 30.

to a few rooms devoted to inspectors. These are placed beneath the medical officers' rooms, and are also served by a staircase at the extreme end of the building, so that the inspectors are in close touch with the medical officers.

As in the other cases which we have dealt with, the principal floor is the first floor (see Fig. 30). Here the main staircase is made a great feature, leading through a handsome circular landing up another flight of steps to a circular hall over the inner hall of the ground floor, architecturally treated with a colonnade. The two radial corridors serve committee-rooms on either side, with a mayor's parlour adjoining the staircase over the Brixton Hill entrance, and a reception-room adjoining those over the Acre Lane entrance. Both these rooms have doors which are exactly opposite to the doors of a long narrow ante-room or crush-room, which gives access through two doors to a large council chamber placed over the rates office in the quietest spot in the whole building, and well lighted from one side. The Town Clerk's offices are ranged *en suite*, off the extension of the corridor on the Brixton Hill side, and so are conveniently placed for serving both committee-rooms and council chamber. As when dealing with other plans the details of the various departments have already been considered, there is no necessity to describe them again, for it will be found that in all essentials there is agreement in such respects. There is proper departmenting and yet proper means of communication, both by staircases and lifts, while for each department the offices which are used by the public are placed near the entrance and those of a more private nature are kept away from them, with strong rooms carefully placed wherever possible away from the external walls.

The second-floor plan (Fig. 31) shows how two galleries are reached over the council chamber, one for the use of members and staff only, being served by the staircase on the Brixton Hill side, while the public gallery is served from the staircase on the Acre Lane side. Between the entrances to these two galleries further stairs lead up on each side to a staff dining-room, which is placed over the ante-room to the council chamber. On the Acre Lane side is also arranged a compact little house for the caretaker,

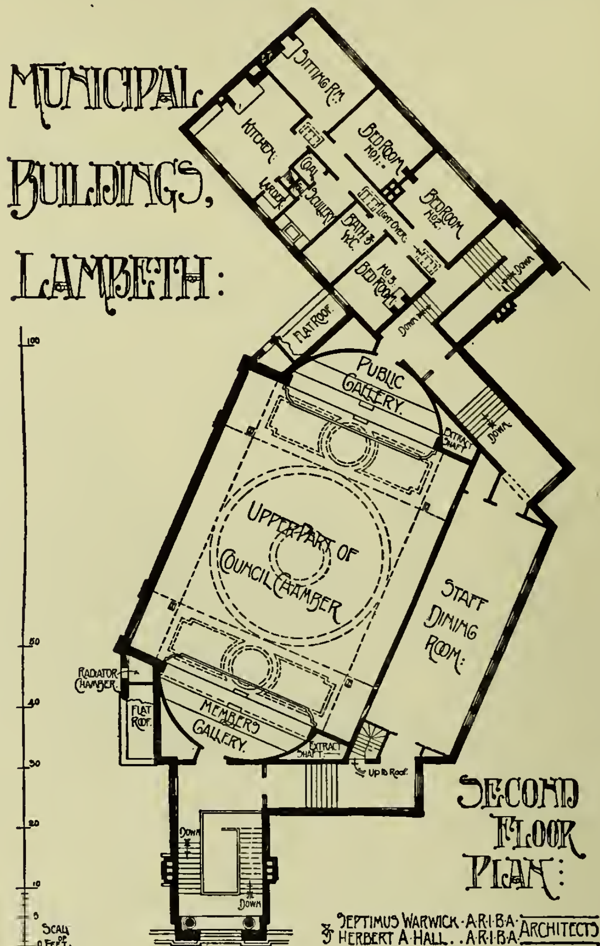


FIG. 31.

who has access with comparative ease to all parts of the building.

The external treatment is shown in Fig. 32, which illustrates all the elevations, and shows the tower, which is carried on the heavily massed masonry seen on plan between the vestibule and the inner hall, and directly over the strong-room in the basement.

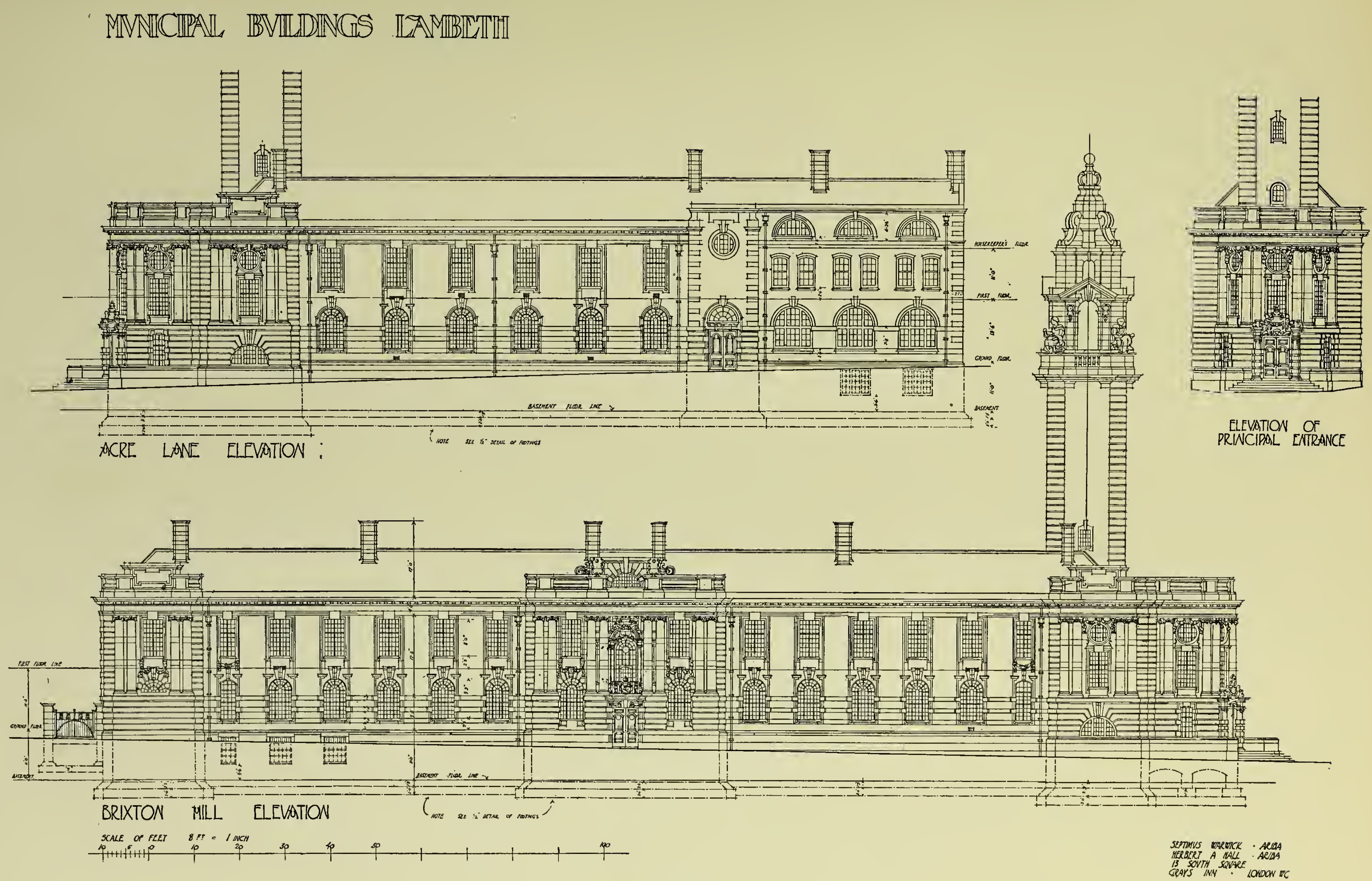

MVNICIPAL BVILDINGS LAMBETH
ACRE LANE ELEVATION :
ELEVATION OF PRINCIPAL ENTRANCE
HOUSEKEEPER'S FLOOR
FIRST FLOOR
GROUND FLOOR
BASEMENT
BASEMENT FLOOR LINE
NOTE SEE ½" DETAIL OF FOOTINGS
BRIXTON HILL ELEVATION
FIRST FLOOR LINE
GROUND FLOOR
BASEMENT
SCALE OF FEET 8 FT = 1 INCH
10 5 0 10 20 30 40 50 100
SEPTIMUS WARWICK · ARIBA
HERBERT A HALL · ARIBA
13 SOUTH SQUARE
GRAYS INN · LONDON WC

FIG. 32.

PART II

STEEL CONSTRUCTION

(*Contributed by P. R. Strong*)

CHAPTER I

ELEMENTARY PRINCIPLES

FORCE is defined as that which changes, or tends to change, the state of rest of a body, or of its uniform motion in a straight line. In the consideration of structures every body should be in a state of rest and should continue in that state. For the present purpose force may be defined as that which tends to change the

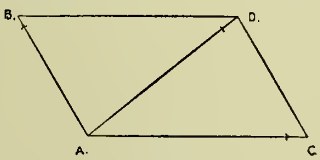


FIG. 33.

state of rest of a body. For the sake of investigation, forces are represented by straight lines, the direction of the force being shown by the direction of the line, and the magnitude by its length.

PARALLELOGRAM OF FORCES.—Let AB and AC (Fig. 33) represent in magnitude and direction two forces acting

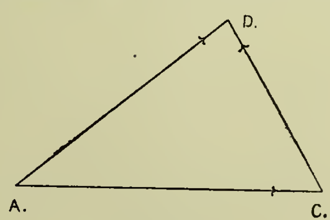


FIG. 34.

upon a point. By completing the parallelogram ABDC the diagonal AD is found, which represents a force equal to the combined effect of the original forces, and is known as the *Resultant* of the forces AB and AC. The same result may be arrived at with less work by describing a triangle (Fig. 34) in which CD = AB in magnitude and direction. AC and CD are known as the *Components* of AD. If AB (Fig. 35) be a given force, its vertical and horizontal components are found by drawing vertical and horizontal lines from its extremities, thus obtaining the two forces AC and CB.

EQUILIBRIUM.—In order that forces may only tend to change the state of rest of a body, all forces acting on that body must be in equilibrium. For equilibrium (1) the algebraic sum of the vertical components of all the forces must be equal to 0; (2) the algebraic sum of their horizontal components must be equal to 0; (3) any tendency to revolve must be met by an equal and opposite tendency.

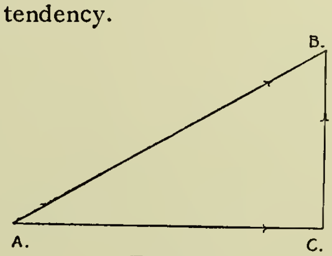


FIG. 35.

TRIANGLE OF FORCES.—In order that there shall be no tendencies to revolve, the line of action of all forces acting upon a body must meet in a point. A little thought will demonstrate the truth of this fact; but, however obvious the fact may appear, it will be well to bear it well in mind, as its application is often extremely useful, while it may very easily be overlooked. In the left-hand diagram of Fig. 36 two forces of 100 lbs. and 200 lbs. are represented in direction. In order to find the magnitude and direction of a single force which will place the series in equilibrium, *ab* is drawn parallel to the force of 100 lbs., and its length is made equal to 100 lbs. on a scale of pounds, while *bc* similarly represents the force of 200 lbs. Then *ac* represents in magnitude and direction the resultant of

the two forces. In order to produce equilibrium, the third force must be equal to *ac* in magnitude, but must be opposite in direction. This force may now be represented on the left-hand diagram by a line parallel to *ca*, and its magnitude may be found by measuring *ca* on the scale of pounds. The line of action of this third force, as stated above, must pass through the common point of the other two forces.

The method of similarly lettering the lines representing the same forces in the two diagrams should be noticed, as it is universal and will be used in all force diagrams to follow. On the left-hand diagram, which gives only the direction and position of forces, the spaces between the forces are lettered; while in the triangle or polygon of forces forming the right-hand diagram the angles are lettered. The result of this is that lines between similar letters represent the same force in each case.

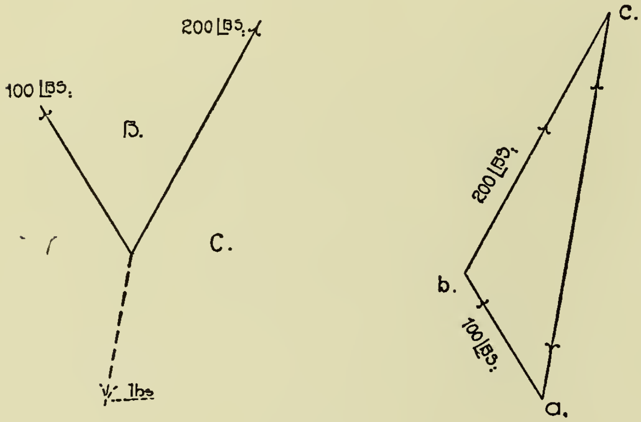
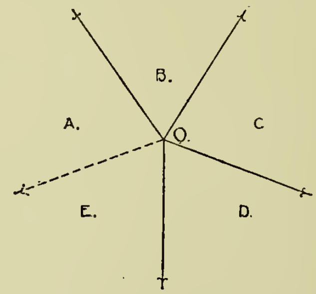


Fig. 36.

Polygon of Forces.—Precisely the same method is employed when any number of forces enter into the problem. In Fig. 37 *ab*, *bc*, *cd*, *de* are drawn parallel to the corresponding forces, between the spaces A and B, B and C, etc., shown acting away from the point O, each force being taken in rotation, and each line, as *bc*, being drawn from the termination of the last, as *ab*. Then the line *ea* represents the force necessary to produce equilibrium; for, by the triangle of forces, *ac* is found to be the resultant of *ab* and *bc*, and may be considered as replacing them. Then *ac* and *cd* will have a resultant *ad*, and again the resultant of *ad* and *de* = *ae*. To place the group of four forces in equilibrium a fifth equal in magnitude to *ae* but opposite in direction must be applied as shown dotted at EA in Fig. 37. The polygon thus obtained is known as a polygon of forces, in which each force represents the resultant of all the remaining forces. The resultant *ac* and *ad*, shown dotted, may be omitted, as they obviously do not affect the result.

Polar and Funicular Diagrams.—It may not always be convenient to produce all forces to their common point of action, as the direction of the forces may be parallel, or so nearly parallel that the point of intersection falls outside the limits of the paper. Unless this common point of action is found the position of the resultant force cannot be fixed by the methods described above. The following describes a device whereby this difficulty is obviated. The construction is of very extensive use, and should be thoroughly noted and understood. Let F_1, F_2, F_3, F_4 (Fig. 38) be a set of forces having a common point of action. Construct a polygon of forces as in Fig. 37. The resultant of the



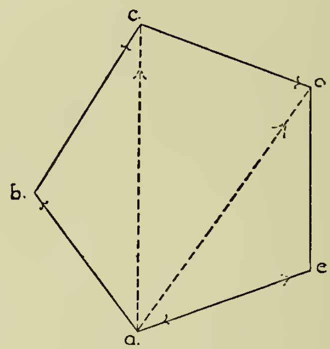


Fig. 37.

forces is then found to be *ae*, which gives the amount and direction of the resultant, but does not give its position with regard to the original forces. Take any convenient point or "pole" *o*. Join *oa*, *ob*, *oc*, etc. Now, by the triangle of forces *ao* and *ob* are components of F_1 (note that the letters are read off in the direction in which the forces act), *bo* and *oc* are components of F_2, and so on. The forces *ob* and *bo* are equal and opposite, and therefore neutralise one another. The only forces which remain unneutralised are the outside ones, namely, *ao* and *oe*, and these are obviously the components of the resultant force *ae*. Take any point in the line of action of F_1 and from this point draw lines parallel to the component part of F_1 as found (namely, *ao* and *ob*). From the point in which the line parallel to *ob* cuts F_2 draw a line parallel to *co*; and similarly draw others parallel to *do* and *oe*. The point of intersection of the lines parallel to *ao* and *oe* is a point in the resultant R, and a line drawn through this point parallel to *ae* will give the position and direction of R.

If the reason of this is not obvious a little more explanation will make it clear. The lines between the

spaces A.O and O.B represent the components of F_1 in direction, and may therefore be assumed to replace F_1; similarly the lines separating the spaces B.O and O.C replace F_2, those between C.O and O.D replace F_3, and those between D.O and O.E replace F_4. As before stated, and as shown in the polar diagram, the forces acting along the lines separating the spaces B.O, C.O, and D.O neutralise one another, and may therefore be eliminated. Thus all the forces F_1, F_2, F_3, F_4 are represented in the two forces separating A from O and

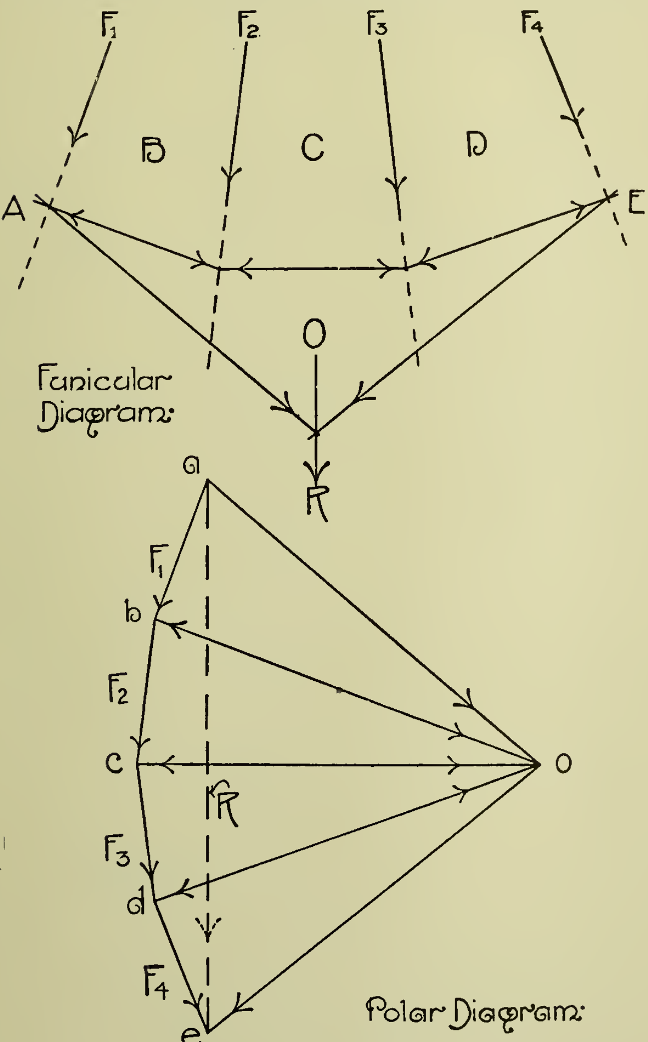


FIG. 38.

O from E, and the resultant of these two forces is R, which is therefore the required resultant.

In the lettering of the funicular diagram in Fig. 38, A represents the external space between F_1 and R. E represents the similar space between F_4 and R. O represents the triangular shaped space within the component parts of the forces.

Had F_1, F_2, etc. been parallel forces, *abcde* would have formed a straight line. The resultant would have been parallel to the forces, and would have been equal to their sum.

TURNING MOMENTS.—It is now necessary to consider those conditions of equilibrium in which a tendency to revolve enters into the problem. Suppose a body AB (Fig. 39) free to revolve about a fixed axis at A. A force of 20 lbs. is applied to the body at a perpendicular distance of twelve feet from the axis. Now, the extent of the turning effect caused by a given force varies directly with the amount of that force, and also directly with its perpendicular distance from the given axis. These two measurements must be combined in order to obtain the turning effect, or turning moment; this is done by simply multiplying the two together, and at the same time originating a unit for this special purpose. The turning moment here becomes 12 feet × 20 lbs. which = 240 foot-lbs.; and the foot-pound is the unit which is employed in this case. Had the distance been given in inches and the force in tons, a unit of inch-tons would have been used for the turning moment. The latter unit is perhaps more largely used than the former in structural design, because of the magnitude of the forces usually dealt with; but any combination of length and weight that may be most convenient for the particular purpose may be employed.

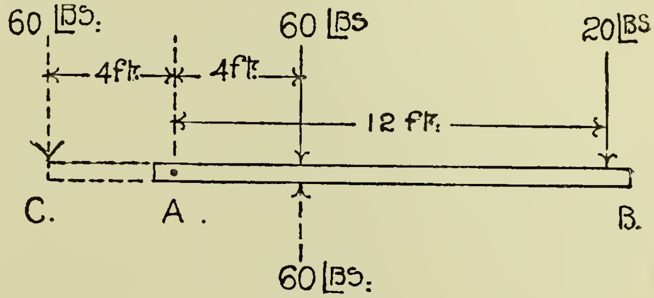


FIG. 39.

Now a force of 60 lbs. acting at 4 ft. from A has its turning moment = 4 ft. × 60 lbs. = 240 foot-lbs. Thus the effect of 20 lbs. at 12 ft. and 60 lbs. at 4 ft. from the axis is seen to be identical; therefore if the force of 60 lbs. be applied in an opposite direction, as shown dotted in Fig. 39, two equal effects are obtained acting in opposite direction, and equilibrium results. If the force of 60 lbs. be applied as shown dotted at C, equilibrium will likewise be established.

It is important to note that the length of the arm is the *perpendicular* distance of the force from the axis. Thus in Fig. 40 the turning moment of W_1 about A = $W_1 \times AD$, and not $W_1 \times AB$. Similarly the turning moment of $W_2 = W_2 \times AE$, and not $W_2 \times AC$.

THE CENTRE OF GRAVITY of a body is that point round which equate all the tendencies to rotate caused by the force of gravity acting upon the various particles of which the body is made up.

For the present subject what is generally required is the centre of gravity of a section of a body. By those who object to the term centre of gravity being applied to a section, which is without thickness and therefore without weight, the term "centroid" is employed, but

"centre of gravity of section" or simply "centre of gravity" is more general.

In parallelograms, circles, and any figure having equal sides and angles the centre of gravity is at its geometric centre. In triangles the centre of gravity is found by bisecting the sides and joining the points thus obtained to the opposite apices, the point of intersection being the required point.

In Fig. 41 the centre of gravity *cg* is found as

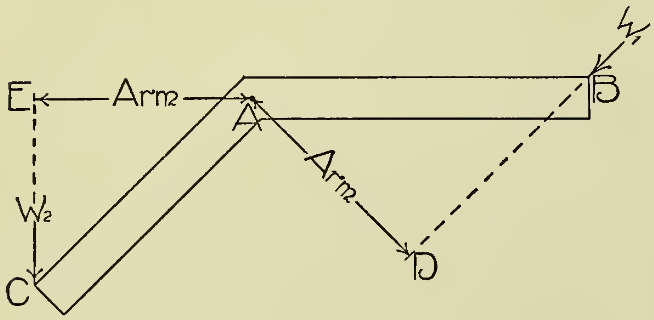


FIG. 40.

follows: DB divides the figure up into two triangles DAB and DCB. The centre of gravity of each is found as described above, and a line EF is drawn joining these points. The figure is then divided up into two fresh triangles, namely, ABC and ADC, by the line AC, GH, joining the centres of gravity of these two triangles, is found as before. The centre of gravity of the whole figure is then given by the point of intersection of EF and GH.

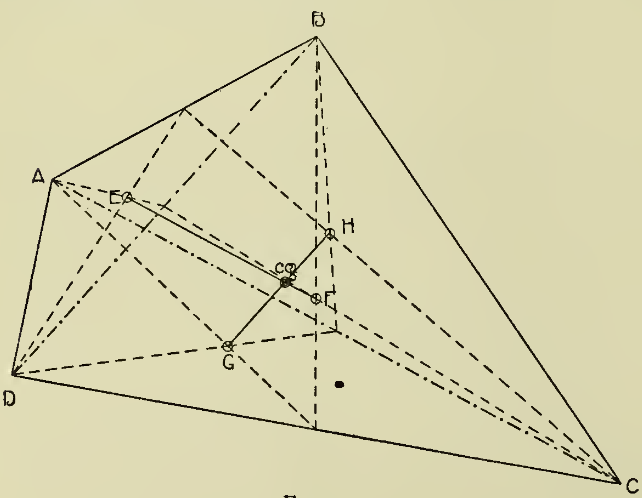


FIG. 41.

In Fig. 42 the centre of gravity of the section of a cast-iron beam is found. The figure being symmetrical about the horizontal axis, its centre of gravity must lie in that axis. The figure is divided up into two rectangles and two triangles, and the centre of gravity of each of these figures is found as described above. Assume each of these figures to exert a downward force from their respective centres of gravity. The amounts of these forces is ascertained by measuring the areas of the figures, and the amounts so found are drawn to scale at *ae*. The position of the resultant of these forces is then found by means of the polar and

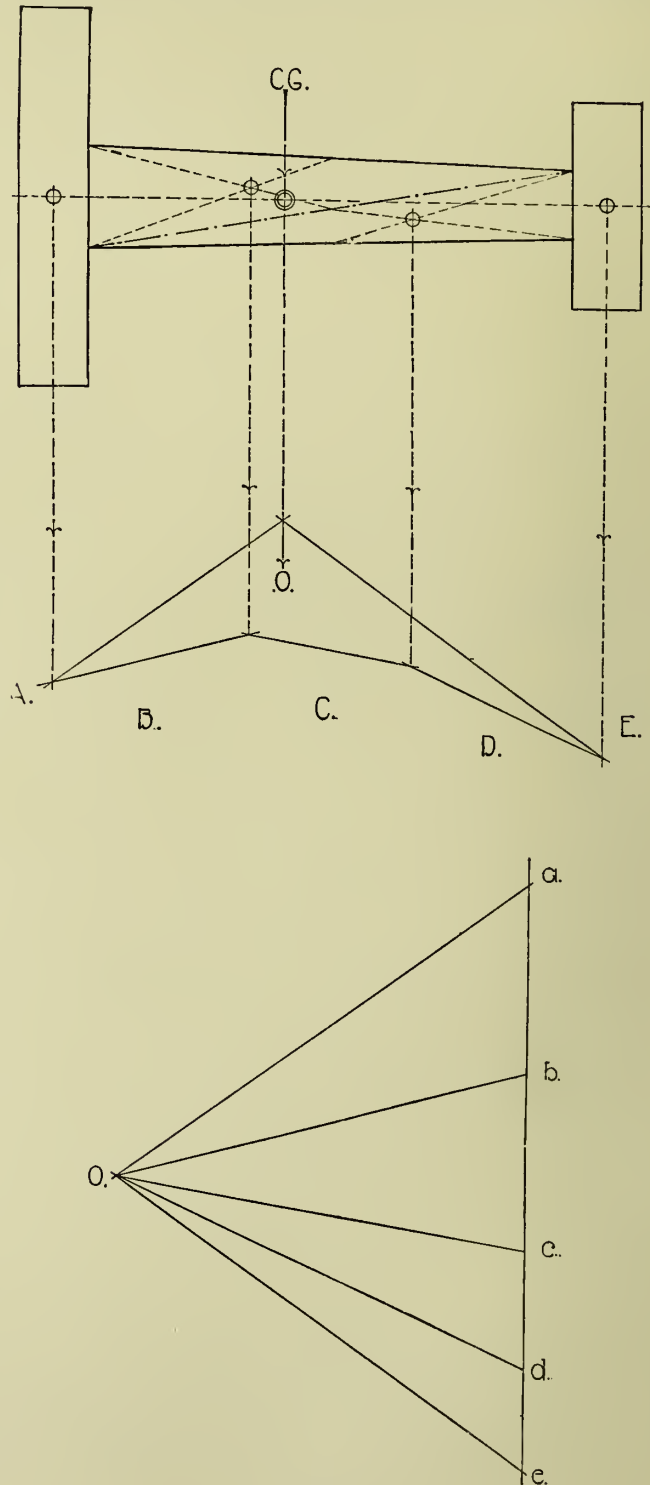


FIG. 42.

funicular diagrams as described in connection with Fig. 38. The point where the line of action of the resultant cuts the horizontal axis is the centre of gravity. With

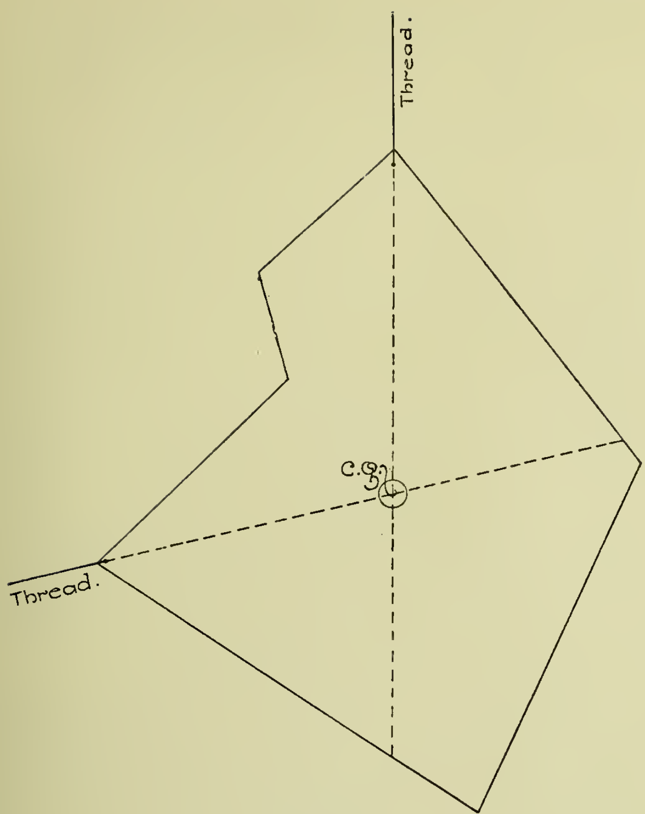


FIG. 43.

the exercise of a little ingenuity the centre of gravity of practically any figure may be found by a combination of the above methods. A practical method of finding the centre of gravity of an irregular figure is as follows. Cut out the figure from a piece of cardboard, and suspend it from a thread at one corner as in Fig. 43. The CG is now known to lie in a line vertically under the thread, and the line is drawn upon the cardboard as shown in a dotted line. The figure is now suspended from another point and another vertical line is drawn. The intersection of the two lines will give the centre of gravity of the figure.

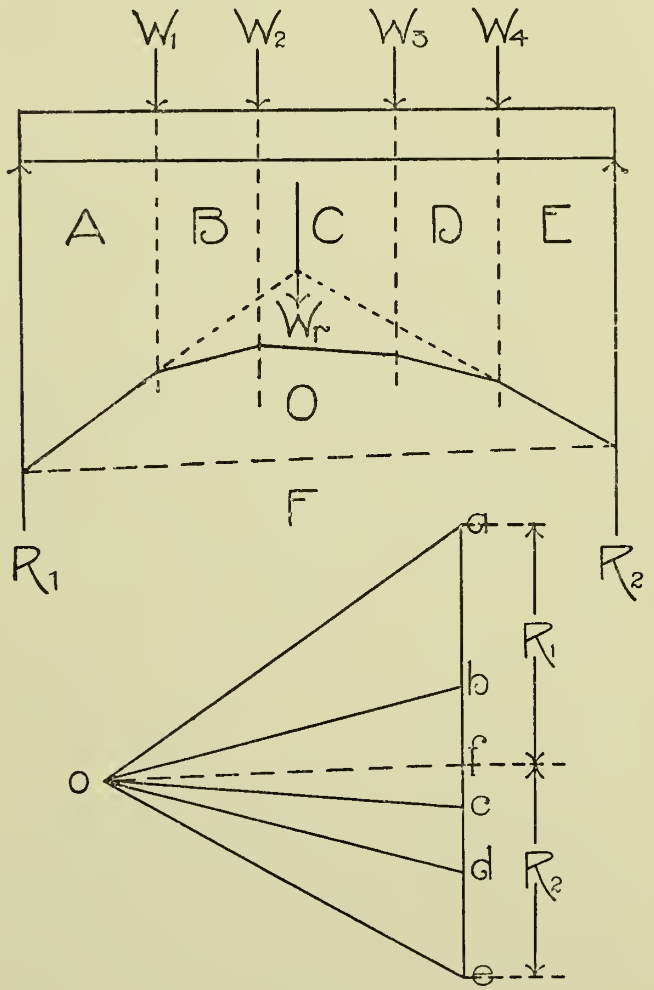


FIG. 45.

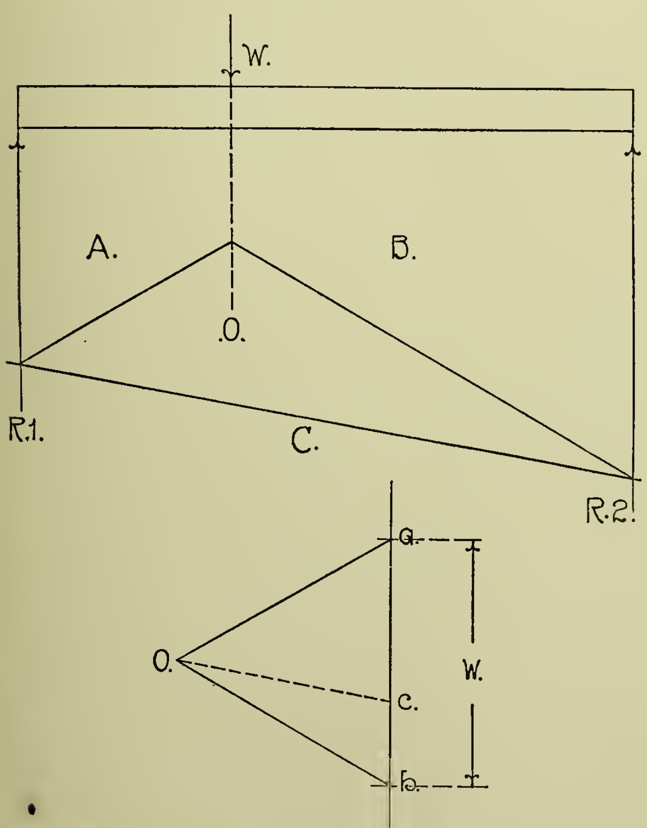


FIG. 44.

REACTION.—When a weight is placed on a support the support exerts an upward force equal and opposite to the force brought to bear on it by the weight. This force, which is only exerted by a body on having an external force applied to it, is called *Reaction*.

Fig. 44 represents a beam which, for the present, may be supposed to be weightless. It supports a weight W, and is carried on supports at either end which exert reactions R_1 and R_2. Then $R_1 + R_2 = W$. To discover how the weight is divided between these two reactions, a slight variation of the construction

described under the head of Polar and Funicular Diagrams (Fig. 38) may be employed. In Fig. 44 *ab* is set off representing W to scale. Take any point *o*, and join *oa*, *ob*. On the line of action of W take a point and draw lines parallel to *oa* and *ob* to cut R_1 and R_2. The lines thus drawn between the spaces A and O and the spaces B and O are components of W; and they are also components of R_1 and R_2 respectively. The other components of R_1 and R_2 must, for equilibrium, be equal and opposite, and must act in the same line; they must also pass through the points already found on R_1 and R_2. The only force which will fulfil these conditions is found by joining the two points on R_1 and R_2; and by drawing *oc* parallel to it its magnitude is found. Thus *co*, *oa* are found to be the components of R_1; therefore *ca* represents R_1. Similarly $R_2 = bc$.

Fig. 45 shows a similar case in which there are four weights. The resultant of the weights is found at W_r, and then the two reactions R_1 and R_2 are found as in the last example. It should always be noted in drawing these diagrams that lines drawn across spaces A, B, C, D, etc. upon the funicular or space diagram are always parallel to *oa*, *ob*, *oc*, *od* respectively in the polar diagram.

The method of determining reaction as described above is that which is chiefly used for the purpose. The following method will probably be found simpler where only a few weights are concerned.

Let a beam 20 ft. long support a weight of 10 lbs. 4 ft. from one end (Fig. 46). Consider the beam free to revolve about B. The weight of 10 lbs. will give a turning moment of 10 lbs. x 16 ft. = 160 foot-lbs. The reaction R_1 at A gives a turning moment about B

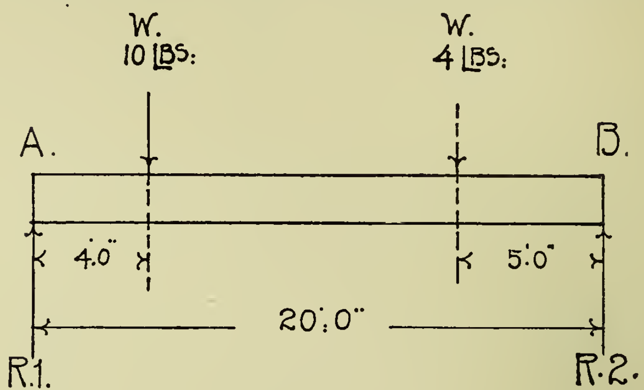


Fig. 46.

of R_1 lbs. x 20 ft. = 20 x R_1 foot-lbs. As the beam is in equilibrium these two moments must equate.

$$\therefore \quad 20 \times R_1 = 160 \text{ foot-lbs.}$$

$$\therefore \quad R_1 = 8 \text{ lbs.}$$

$$\text{The sum of } R_1 \text{ and } R_2 = 10 \text{ lbs.}$$

$$\therefore \quad R_2 = 2 \text{ lbs.}$$

Precisely the same result will be arrived at if A be considered as the axis of rotation. The beginner should satisfy himself of the truth and the reason of this. If a weight of 4 lbs. be added 5 ft. from B, as shown dotted in Fig. 46, the equation for R_1 becomes

$$20 \times R_1 = (10 \times 16) + (4 \times 5) = 160 + 20 = 180.$$

$$\therefore \quad R_1 = 9 \text{ lbs.},$$

$$\text{and } R_2 = 10 + 4 - 9 = 5 \text{ lbs.}$$

CHAPTER II

STRESSES IN BEAMS

Strain.—If a body be acted upon by an external load a change of form takes place in it. This change of form is called *Strain*. If the load acts in a direction directly towards the point of support of the body, the strain is one of compression; or if the load acts directly away from the point of support, it is one of extension. If the load acts in a direction which is not in line with the resistance of the support, the strain takes the form of bending.

Stress is the resistance of a body to change of form. The two terms *strain* and *stress* must not be confounded with one another. Strain is measured lineally, whilst stress is measured in terms of force. In stable structures of steel or iron strain is proportional to stress (see page 65).

Stresses produced by Bending Moments.—Fig. 47 represents a cantilever with end load W. Considering the effect of W at the section at A, the turning or bending moment produced by $W = W \times l$. In order that the beam may remain in its horizontal position, there must be an equal and opposite turning moment acting at A. The forces which go to form this turning moment, or "Moment of Resistance," are indicated at F_t and F_c, each being at a distance of $\frac{d}{2}$ from the central point of the section A. Thus this moment of resistance $= F_t \times \frac{d}{2} + F_c \times \frac{d}{2} = F_t \times d$, or $F_c \times d$, F_t and F_c being equal in amount. If we know the length of d we can ascertain the amount of F_t and F_c from the equation—

$$F \times d = W \times l, \text{ or } F = \frac{Wl}{d}.$$

F_t is a tensional stress, and F_c a compressional stress.

Fig. 48 represents a beam with central load W. R_1 and R_2 are therefore each equal to $\frac{W}{2}$. R_1 exerts a bending moment of $\frac{W}{2} \times \frac{l}{2}$ at the section of the beam immediately under W, and equating with the stresses produced in the beam—

$$F \times d = \frac{W}{2} \times \frac{l}{2} = \frac{W \times l}{4}, \text{ or } F = \frac{Wl}{4d}.$$

The principles governing the stresses in the beam are seen to be the same as those which we have considered in the case of the cantilever; however, in the beam the top portion is in compression, while in the cantilever it is in tension.

Disposition and Intensity of Stress.—From the last paragraph it is seen that while one portion of a beam is in compression the opposite portion is in tension, and that for equilibrium these two stresses must be equal. It is obvious that between these two opposing forces there must be a neutral layer where there is neither compression nor tension.

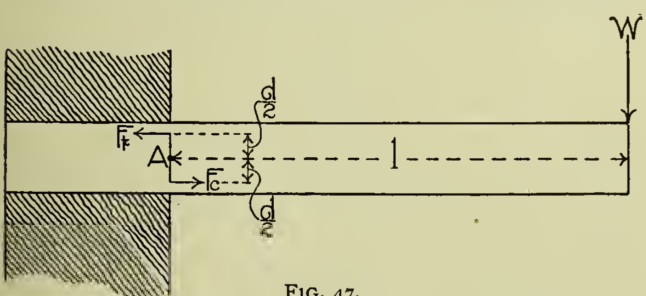


Fig. 47.

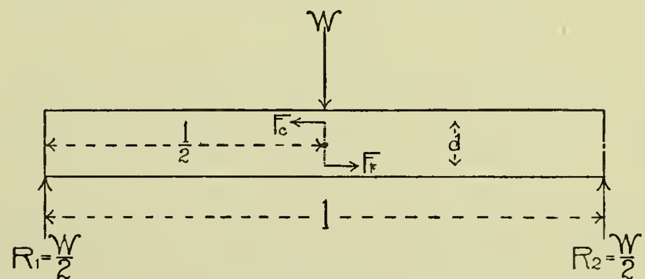


Fig. 48.

In Fig. 49, ABCD is the elevation of a fixed cantilever, EF being its neutral plane. AGHD represents the same beam bent as shown, the neutral plane taking the form EK. Now, as there is no stress at the neutral plane its length will be *constant*. ∴ EK = EF. The end BC assumes a position GH, pointing towards the centre of curvature. Had the lengths of AB and DC remained unaltered, BC would have taken a position LM. Thus the layer AB has been strained to an extent equal to LG, and DC has been strained to the extent of HM, the former being extension and the latter compression.

The distribution of the strain is shown by the shaded areas, and it is seen to be proportional in each layer to the distance of that layer from the neutral axis. Referring to the definition of stress, it is stated that strain is proportional to stress. Therefore the intensity of stress in each layer is proportional to its distance from the neutral plane.

Considering the section on one side of the neutral axis as divided up into horizontal layers of unit thickness, whose distances from the neutral axis are y_1, y_2, y_3, etc., and whose breadths are b_1, b_2, b_3, etc., while f is the intensity of stress per unit of area at unit distance from the neutral axis; then the intensity of stress in the layer at distance y_1 from the neutral axis $= y_1 f$; and the total stress in that layer $= b_1 y_1 f$. The total stresses in other layers are similarly $b_2 y_2 f$, $b_3 y_3 f$,

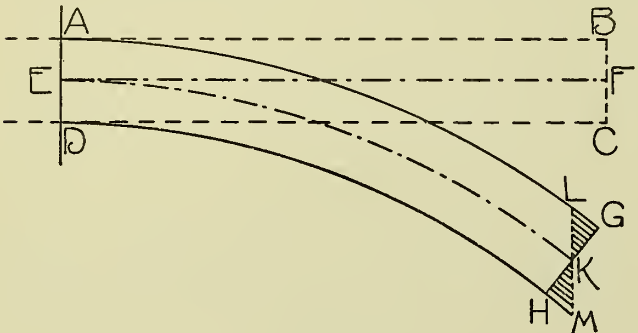


Fig. 49.

etc., and the total stress in the half sections $= f(b_1 y_1 + b_2 y_2 + b_3 y_3 + \text{etc.})$.

If now the opposite half of the section be similarly treated, denoting breadth of layers by b'_1, b'_2, b'_3, etc., and their distances from the neutral axis by y'_1, y'_2, y'_3, etc., the total stress in this half $= f(b'_1 y'_1 + b'_2 y'_2 + b'_3 y'_3 + \text{etc.})$.

As mentioned above, the total stresses on either side of the neutral axis must be equal.

$$\therefore f(b_1 y_1 + b_2 y_2 + b_3 y_3 + \text{etc.}) = f(b'_1 y'_1 + b'_2 y'_2 + b'_3 y'_3 + \text{etc.}).$$

$$\therefore (b_1 y_1 + b_2 y_2 + b_3 y_3 + \text{etc.}) = (b'_1 y'_1 + b'_2 y'_2 + b'_3 y'_3 + \text{etc.}),$$

which is an expression given by the definition of the centre of gravity. Thus it is shown that the neutral axis passes through the centre of gravity of a section.

Fig. 50 is the section of a rectangular beam. Let the intensity of stress in the outer layer AB be represented by the width of that layer. The intensity of stress in each successive layer decreases uniformly to zero at the neutral axis. Thus the intensity of stress in each layer is given by the length of that portion of the layer which is cut off between the lines AD and BC, and the shaded portion of the figure shows the intensity of stress in each layer of the section. As the beam here shown is rectangular and the width of each layer is the same, the total stress in each layer is proportional to the intensity of stress in that layer, so that if AB be taken as representing the total stress in the outer layer the lengths cut off between AD and BC will give the total stress in each layer, and the shaded portion will then give the total stress in the whole section.

In Fig. 51 the section of a rolled steel joist is shown. As before, the intensity of stress in each layer is given by a horizontal line passing through the layer in question, and bounded by the lines AD and BC. For instance, the intensity of stress at any layer G is to the intensity of stress at AB, as HK is to AB.

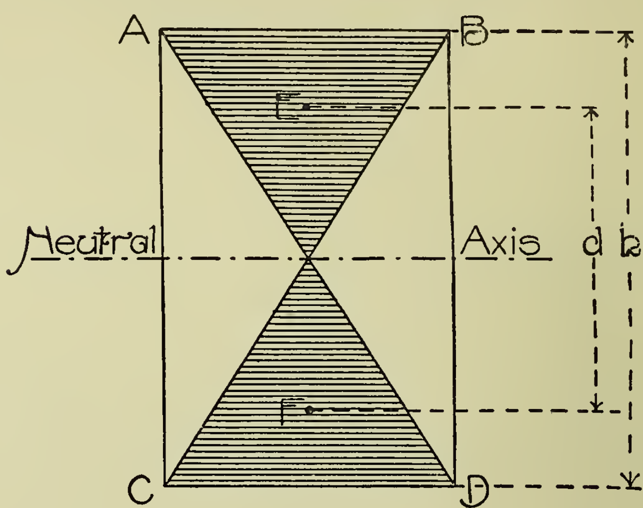


Fig. 50.

If the length of AB be taken as representing the total stress in that layer, the total stress in any other layer may be found by referring its width to the

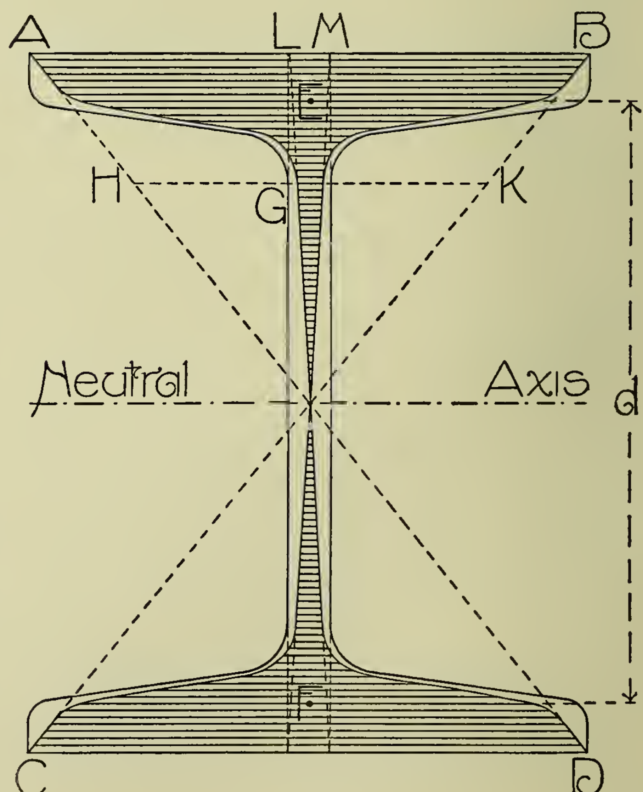


Fig. 51.

line AB, and by joining the points thus found to the centre of gravity of the section. Thus the total stress in the web is found by marking off its width on AB at LM. LM gives the total stress in a portion of

AB equal to the width of the web, and by joining points L and M to the centre of gravity of the section the total stress in each layer of the web is found. The shaded area found in this way represents the total stress in the beam's cross section in terms of the stress in AB.

In Fig. 52 the stress area or "equivalent figure" of the section of a cast-iron beam is found. Here, as before, AB is taken as representing the total stress in the outer layer, and all widths on this side of the neutral axis are referred to this line. The line GH is drawn at the same distance from the neutral axis as is AB. If the section were to extend down to GH, the intensity of stress would there be equal to the intensity

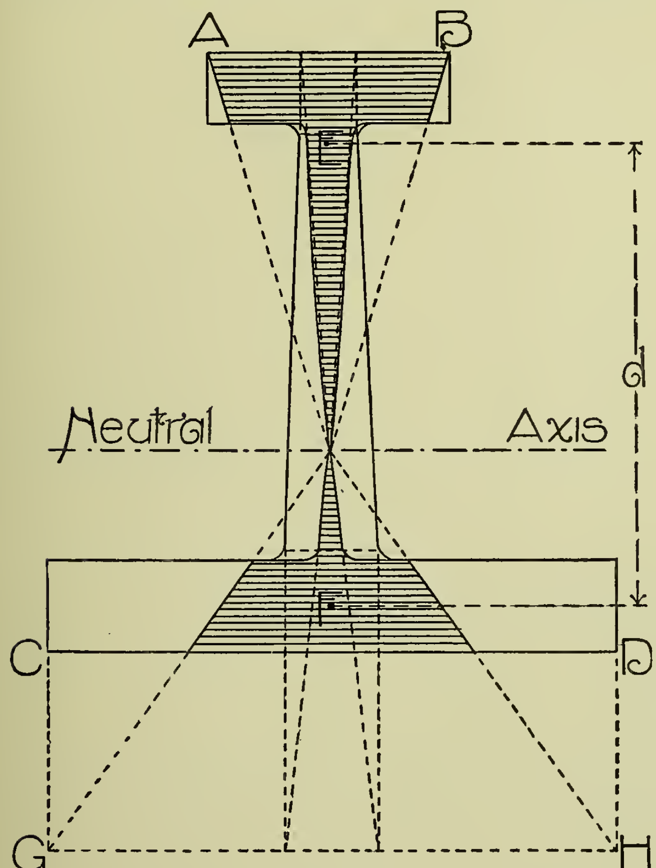


Fig. 52

at AB. Thus if GH be made equal to CD, the total stress in that line will be represented by its length, and the actual total stress at CD is found by joining G and H to the centre of gravity of section. Thus to obtain the total stress in any layer on this side of the neutral axis, the widths must all be referred to the line GH. As the web in this case is not of the same width throughout, the widths of several layers in its height must be referred to AB or GH in order to obtain the bounding curve, as indicated in the figure.

Fig. 53 gives the stress area for the same section as that given in Fig. 52; but in this case the line CD is taken as representing the stress in that layer, and the other reference line GH is drawn at an equal distance from the neutral axis.

The only difference in the stress areas of Figs. 52 and 53 is that the stresses are drawn to a larger scale in the latter than in the former.

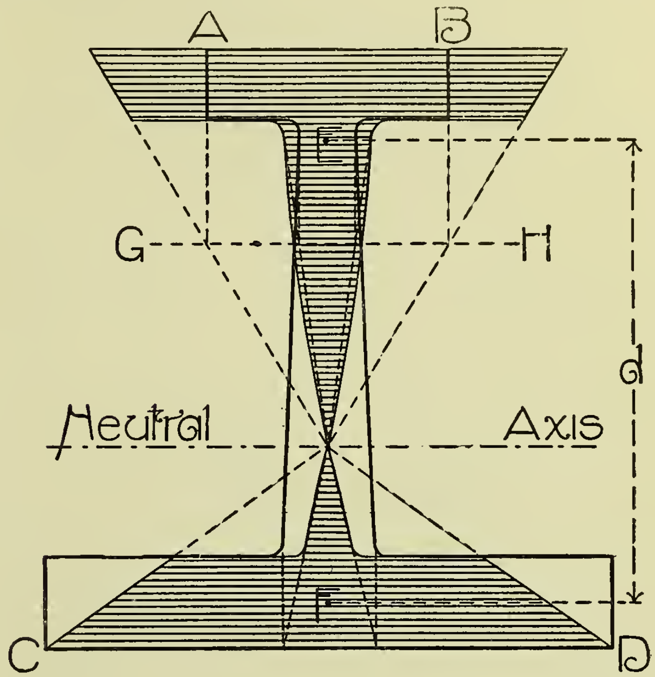


Fig. 53.

Referring again to Fig. 50, it is evident that the section is far from being economical, and that greater economy is obtained when as little material as possible is placed near the neutral axis, as in Fig. 51. There

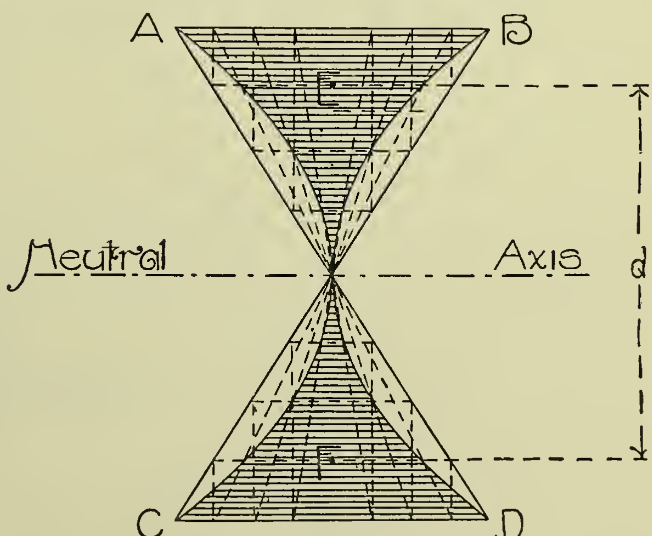


Fig. 54.

sometimes exists an impression that if the unshaded portion of the section shown in Fig. 50 were removed the beam would be as strong as before. This is of course untrue, for the intensity of stress in each layer remains the same while the total stress borne by each

layer is reduced in proportion to that width of the layer. The section thus reduced is, however, far more economical of material than the rectangular section; its stress area and the necessary construction are shown in Fig. 54.

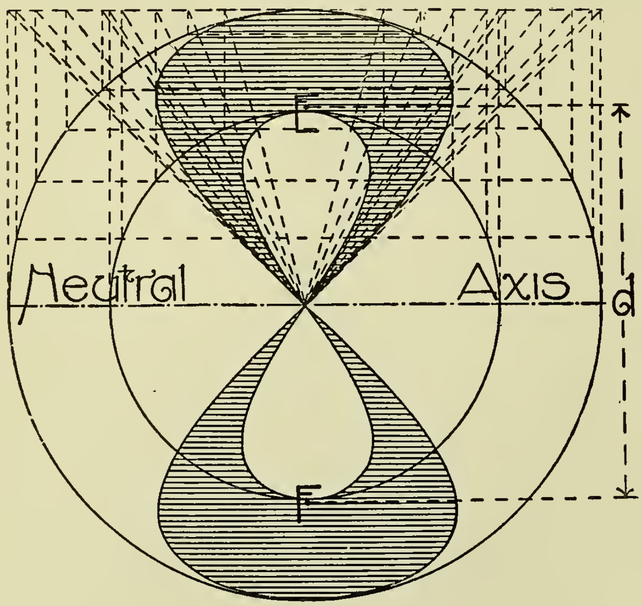


Fig. 55.

The stress area for a hollow circular section is shown in Fig. 55.

Fig. 56 shows the section of one bay of a form of steel trough flooring with its equivalent figure, which by the dotted line. The equivalent figure of this dotted section is now found in the manner already explained.

Modulus of Section.—The two halves of the stress area on either side of the neutral axis, as in Figs. 50 to 56, are in every case equal to one another. The total stress on each half of the section may be taken as acting at the centre of gravity of each half of the stress area, represented by the points E and F in Figs. 50 to 56. The sum of each half of the stress area, multiplied by the distance of its centre of gravity from the neutral axis, or, which amounts to the same thing, the area of one-half of the stress area multiplied by the distance between the two centres of gravity d, is called the "Modulus of section," and is generally denoted by the letter Z.

Moment of Resistance. — Let f denote the unit intensity of stress at the outer layer of the section, or the "extreme fibre stress"; that is, the greatest stress per square inch that the material is called upon to bear. The moment of resistance mentioned in connection with Fig. 47 now becomes Z $\times f$.

Referring again to Fig. 50, it is obvious that half the stress area $= \frac{b \times h}{4}$, where b and h denote the breadth and depth of the beam, and that the arm $d = \frac{2h}{3}$.

Therefore the modulus of section in this case $= \frac{bh}{4} \times \frac{2h}{3} = \frac{bh^2}{6}$. It is usual to write this as $\frac{bd^2}{6}$, and we will therefore write it so in future, but the reader must not let himself be confused by the use of the letter "d"

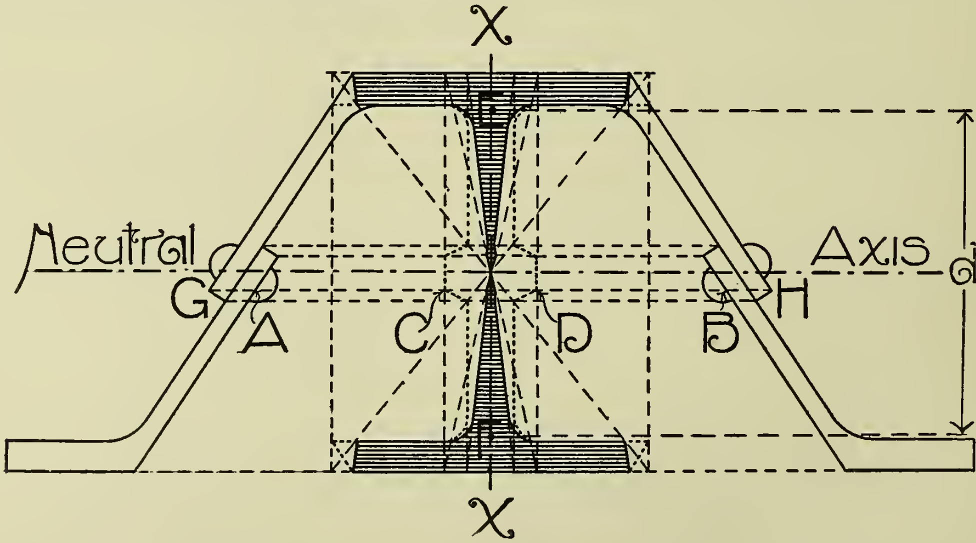


Fig. 56.

may be found by the method already explained. It is more convenient, however, to transfer the width of steel in each layer of the section to the axis XX. Thus in the layer GH the points C and D are marked off at distances on either side of XX equal to GA or BH. In a similar manner a number of other points are found, which mark out the section of I form indicated to signify both the effective depth and total depth of a beam's section.

The moment of resistance of the above section = Z . f $= f . \frac{bd^2}{6}$.

The modulus of section of the sections, Figs. 51 to 56, though not so easily expressed, are ascertained by

measuring half the stress area and multiplying this by the *effective* depth, or "d" as figured thereon.

It would appear from the above that it is possible to calculate the resistance to bending of any section, so long as we know the ultimate strength in direct tension and compression of the material in use; but experiment proves that the strength, as arrived at by the formula, is in some cases considerably less than what is actually the case. This is probably partly accounted for by the fact that the outer layers derive assistance from the adjacent layers. Also, the formula starts with the assumption that stress and strain are proportional, and it therefore fails when it is used to find the breaking resistance (see page 65).

Therefore, instead of employing a factor f as found by direct compressive or tensile experiments, it should be modified to conform to the results of experiments made in resistance to bending. "f" thus modified may be written f_0, which is called the "MODULUS OF RUPTURE," and a new value must be found for it when calculating the strength of circular or other shaped beams. In the section shown in Fig. 51 the metal is nearly all disposed where it will receive its maximum amount of stress, and the outer layers receive very little assistance from layers nearer to the neutral axis. For this reason the formula, moment of resistance $=fZ$, is very near the truth, and is universally used in calculating the strength of these sections.

MOMENT OF INERTIA, "I."—Again consider the beam section in Fig. 50. The moment of resistance has been shown to be equal to $f\frac{bd^2}{6}$, f being the stress intensity at the outer layer of the section; that is, at a distance of $\frac{d}{2}$ from the neutral axis. If p = the stress intensity at unit distance from the neutral axis, $f = \frac{d}{2}p$. Inserting this value of f in the formula for moment of resistance, it becomes $\frac{bd^2}{6}\cdot\frac{d}{2}\cdot p = \frac{bd^3}{12}p$. $\frac{bd^3}{12}$ is the moment of inertia "I" for a rectangular section. In this case the moment of resistance evidently $=\frac{bd^3}{12}\times\frac{f}{\frac{d}{2}} = \mathrm{I}\times\frac{f}{\frac{d}{2}} = \frac{\mathrm{I}}{\frac{d}{2}}\cdot f$, and in all cases moment of resistance $= \frac{\mathrm{I}}{y}\cdot f$, and modulus of section $\mathrm{Z} = \frac{\mathrm{I}}{y}$, where y = the distance from the neutral axis of the layer in which the stress intensity $=f$. In the case of a symmetrical section $y = \frac{d}{2}$.

The value of I for a section such as is shown in Fig. 57 $=\frac{\mathrm{B}\cdot\mathrm{D}^3}{12} - 2\frac{b\cdot d^3}{12} = \frac{\mathrm{BD}^3-2bd^3}{12}$. I may be found in another way, thus—

$$\mathrm{I} = \frac{\mathrm{B}\cdot\mathrm{D}^3}{12} - \frac{\mathrm{B}d^3}{12} + \frac{td^3}{12} = \frac{\mathrm{B}}{12}(\mathrm{D}^3-d^3) + \frac{td^3}{12}.$$

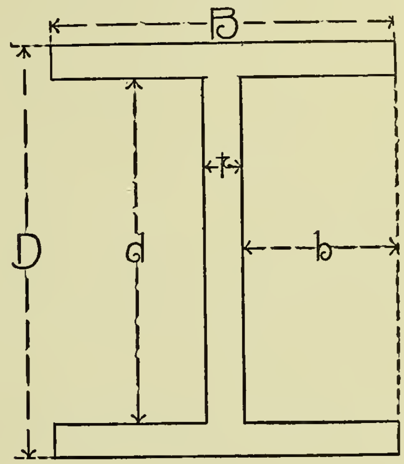


FIG. 57.

By dividing it up into rectangles the value of I for any symmetrical figure may be found by the last method. Thus in Fig. 58—

$$\mathrm{I} = \frac{\mathrm{B}_1}{12}(\mathrm{D}_1^3 - d_1^3) + \frac{\mathrm{B}_2}{12}(\mathrm{D}_2^3 - d_2^3) + \frac{\mathrm{B}_3}{12}(\mathrm{D}_3^3 - d_3^3) + \frac{td_3^3}{12}.$$

The value of I for the section shown in Fig. 60 about axis $\mathrm{XX} = 2\mathrm{I}_1 + \frac{\mathrm{B}}{12}(\mathrm{D}^3 - d^3)$, where I_1 = the moment of inertia of the H-shaped section.

I may be defined as the sum of the products of the areas into which the section may be divided, multiplied by the square of their respective distances from the neutral axis. These areas are supposed to be so small

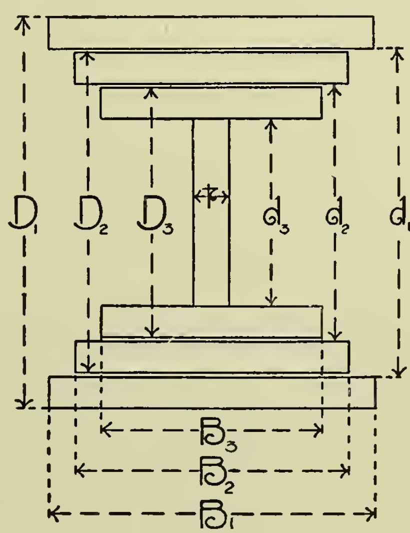


FIG. 58.

that the intensity of stress in them is constant throughout each. Then, if the areas be denoted by a_1, a_2, a_3, etc., and their distances from the neutral axis by y_1, y_2, y_3, etc., $\mathrm{I} = a_1y_1^2 + a_2y_2^2 + a_3y_3^2 +$ etc. = the sum of the $a\cdot y^2$'s, which is generally denoted thus, $\mathrm{I} = \Sigma ay^2$. In the value of I given above, $\frac{bd^3}{12} = b\cdot d \times \frac{d^2}{12}$; $b\cdot d$ = the area of the cross section, and is therefore equal to Σa,

while $\frac{d^2}{12}$* may be considered as the average of the values of y^2.

The moment of inertia of a figure about an axis XX

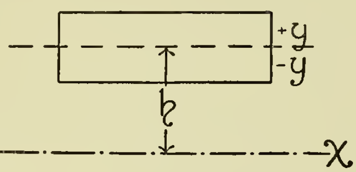


Fig. 59.

(Fig. 59), at a distance h from its centre of gravity, may be considered as follows.

The distance of each elementary area "a" from axis $XX = h+y_1$, $h+y_2$, $h+y_3$, etc. on the side remote from XX, and $h-y_1$, $h-y_2$, $h-y_3$, etc. on the side nearer XX.

Thus the required moment of inertia—

$$= \Sigma a(h \pm y)^2$$
$$= \Sigma a(h^2 \pm 2hy + y^2)$$
$$= \Sigma ay^2 + \Sigma 2ah(\pm y) + \Sigma ah^2.$$

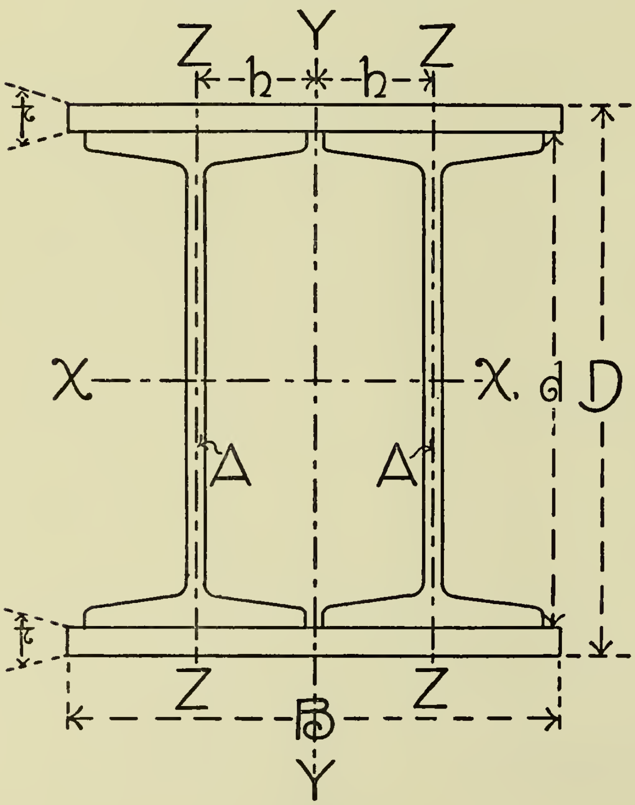


Fig. 60.

The quantity—

$\Sigma ay^2 = I_1 =$ the moment of inertia of the figure about its neutral axis, which is parallel to XX.

$\Sigma 2ah(\pm y) = 0$; for the positive and negative values of y cancel one another.

$\Sigma ah^2 = h^2 \Sigma a = h^2 A$; for $\Sigma a =$ the total area of the section $= A$.

$\therefore$ The moment of inertia about $XX = I = I_1 + Ah^2$.

* This is the Radius of gyration (see page 87).

For the section shown in Fig. 60, where the moment of inertia of each H-shaped section about

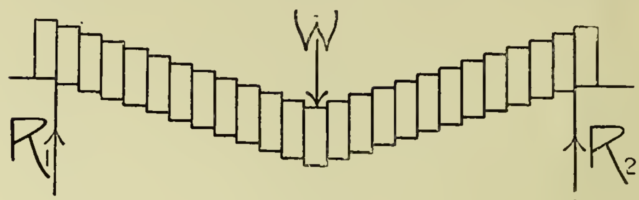


Fig. 61.

axes $ZZ = I_1$, and whose areas $= A$, I about axis YY $= 2(I_1 + Ah^2) + 2 \cdot \frac{tB^3}{12}$.

For solid circle $I = \frac{\pi}{4}R^4 = .7854R^4$.

For hollow circle $I = .7854(R^4 - r^4)$, where $R =$ external radius and $r =$ internal radius.

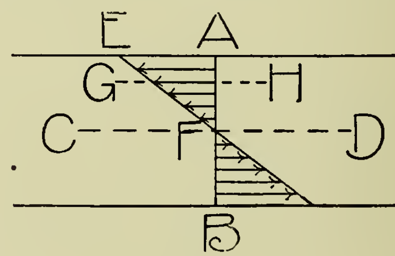


Fig. 62.

Vertical Shear.—Hitherto we have only considered the longitudinal stresses produced in beams by their

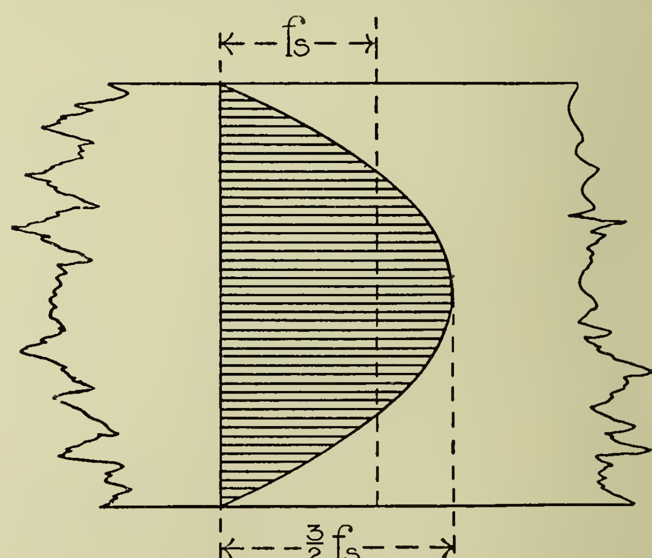


Fig. 63.

resistance to bending. The stress set up in conveying the direct downward effect of the load to the supports is called shear. The tendency throughout the beam is one in which every vertical layer tends to slide upon the next. This is illustrated in Fig. 61. This tendency on either side of the load is equal in amount to that portion of the load which is carried to the supports.

Thus in Fig 61, between R_1 and W the total shear stress at each point in this length $= R_1$ and the intensity of this stress $= R_1$ divided by the area of section of the beam. Between W and R_2 the total shear stress $= R_2$.

HORIZONTAL SHEAR STRESS.—Fig. 62 shows the disposition of the compressional and tensional stress at a vertical section AB of a rectangular beam. It is obvious that these gradually decreasing and eventually opposing forces must set up a shearing stress between the horizontal layers of the beam. At the neutral plane this horizontal shear is caused by opposing forces of an extent represented by the triangle AEF. At a layer GH the shear is caused by a force of extent represented by AEGH. Thus it is seen that the horizontal shear is greatest at the neutral axis, decreasing to zero at outer edge of the beam. The distribution of this stress in a rectangular beam is parabolic, as indicated in Fig. 63. The maximum shear at the neutral axis $= \frac{3}{2} f_s$, f_s being the average shear. The average horizontal shear per unit of length = the average vertical shear per unit of depth (see page 82).

CHAPTER III

BEAMS AND BENDING MOMENTS

THE DETERMINATION OF BENDING MOMENT AND SHEAR.—It has been shown that the stress produced in a beam or cantilever is proportional to the bending moment. If it be intended to use a beam, such as a rolled steel joist, which is of uniform section throughout its length, a section must be employed which is strong enough to resist the stresses produced in it by the greatest bending moment which will be brought to bear upon it. If it be intended to use a girder composed of simple rolled steel sections riveted together, the amount of metal in the girder, or the depth of the girder, may be apportioned to meet the greatest stress that will be brought upon each point in its length. It is then necessary to know the bending moment at every point throughout the beam, and for this purpose diagrams are drawn as shown in Figs. 64 (A to P), in which the vertical ordinates show the BM throughout the length of the beam. Diagrams showing the shear in each case are also given.

A cantilever with end load is illustrated at "A," and the maximum bending moment obviously $= Wl$. Wl is set down, perpendicularly to a line representing the length of the cantilever, to a scale of foot-lbs., foot-tons, etc., as the case may be. The BM at any other point at distance x from $W = Wx$. Thus the BM is seen to vary as x, and consequently the line bounding the values of BM at every point in the beam's length is a straight line. The weight W is carried from layer to layer to the abutment, and consequently the shear is uniform throughout, and $= W$. This is drawn to a scale of weights.

"B" represents a cantilever with a uniformly distributed load equal to w per unit of length, the total load being equal to W. This weight W may be considered as acting at the centre of the length of the beam, and the maximum BM therefore $= W \times \frac{l}{2} = \frac{Wl}{2}$. At any point at distance x from end of cantilever the $BM = x \cdot w \times \frac{x}{2} = \frac{x^2 w}{2}$. Here the BM varies with x^2, and the bounding curve is consequently a parabola. The weight carried from layer to layer in the cantilever's length is seen to increase uniformly as the abutment is approached, giving a shear diagram as illustrated.

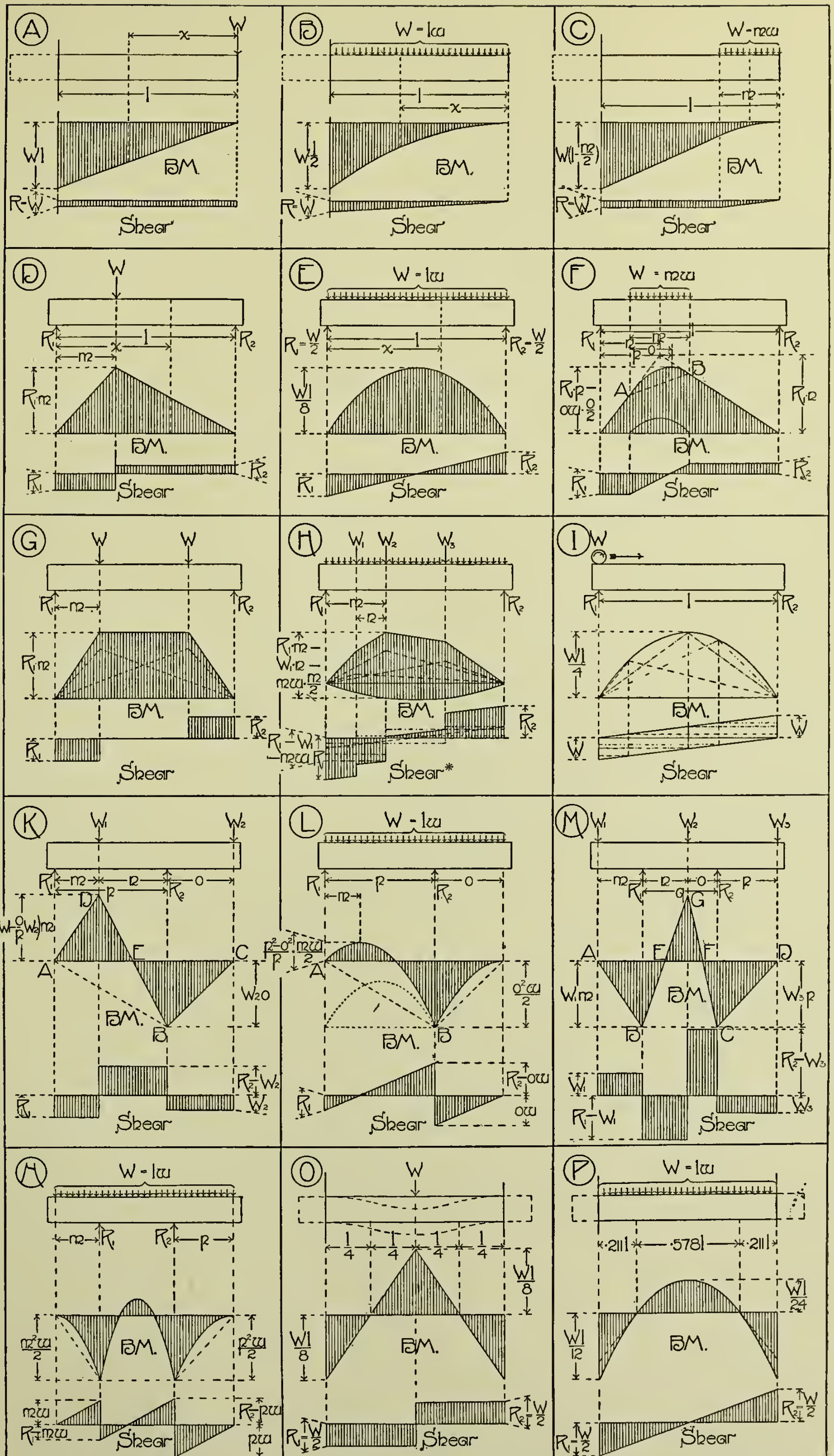
"C" shows a cantilever with distributed end load. The working out of diagrams for this is an obvious combination of the two previous cases.

At "D" a beam supports a single load W. Reaction R_1 may be found from the equation $R_1 \times l = W(l-m)$, as already described. To discover the BM at any point, consider the turning effect of all forces between this point and one end of the beam. Thus maximum $BM = R_1 \times m$, and BM at any point of distance x from $R_1 = R_1 x - W(x-m)$. If W acts at the centre of the beam, the $BM = \frac{W}{2} \times \frac{l}{2} = \frac{Wl}{4}$. The shear between the load and either abutment = that portion of the load which is carried to that abutment, and is consequently equal to the reaction at that abutment.

It should be noticed that the BM diagram is here "D" shown above the line, while in the three preceding cases "A," "B," and "C" they were drawn below the line. The usual custom is to draw the diagram on the side at which compression takes place. It is also seen that part of the shear diagram is drawn above the line and part below. The custom in this is as follows. Considering any section in the length of the beam, if the shearing tendency on the right of this section be an upward tendency the diagram is drawn above the line; if it be a downward tendency the diagram is drawn below the line.

At "E," R_1 and $R_2 = \frac{lw}{2}$. The maximum BM occurs at the centre, and, taking the moments caused by loads between this point and one abutment, as in the last example, the $BM = \frac{lw}{2} \times \frac{l}{2} - \frac{lw}{2} \times \frac{l}{4} = \frac{l^2 w}{8} = \frac{Wl}{8}$, as $lw = W$. In the above equation $\frac{lw}{2} \times \frac{l}{2} =$ the turning moment of the reaction at the abutment, and $\frac{lw}{2} \times \frac{l}{4}$ is the turning moment of that part of the distributed load which lies between the central point and the abutment, $\frac{lw}{2}$ being the amount of this load and $\frac{l}{4}$ the arm. At any point of distance x from R, the $BM = \frac{W}{2} \times x - x.w \times \frac{x}{2} = \frac{Wx}{2} - \frac{wx^2}{2}$, and this is the equation for a parabola.

At "F," $R_1 \times l = W \times (l-n)$ thus obtaining the value of R_1 and R_2; for $R_2 = W - R_1$. The BM diagram may be drawn as follows. Considering W to be concentrated at the centre of m, draw the diagram as in "D." On the base line a parabola is set up for a beam of length m, as shown in "E." Ordinates to this parabola are



DIAGRAMS SHOWING THE DISTRIBUTION OF BENDING MOMENT & SHEAR IN VARIOUS CASES.

*FOR THE SAKE OF CLEARNESS THIS SHEAR DIAGRAM IS DRAWN TO DOUBLE THE SCALE OF THE OTHER CASES

FIG. 64.

now set up from the line AB, the whole curve being as shown in the figure.

At "G," the BM curve is obtained by drawing diagrams for each load separately, and summing the ordinates to the curves thus found. The two weights being equal in this case, the beam is seen to be free from shear between them.

At "H," the BM curve is obtained for W_1, W_2, and W_3 in the same way as in the last case. The curve for the distributed load is here drawn below the line, contrary to the usual custom, but it has the advantage of saving labour and giving greater accuracy. The BM is given by the perpendicular between the extreme lines. If preferred, the parabola may be drawn above the line, its ordinates being added to the ordinates of the concentrated loads. The shear diagram is here arrived at by first drawing the diagram for each load separately, then adding the ordinates on one side of the line and subtracting those on the other side. Another way to consider the problem is as follows. On the extreme right the shear $= R_2$. At W_3 it has been decreased by the amount of the distributed load between W_3 and R_2. On the left of W_3 the shear becomes $R_2 - W_3 -$ the above-mentioned distributed load. The shears at W_2, W_1, and R_1 are found in the same way, negative quantities being drawn below the line.

"I" shows the diagrams for a rolling load. As the load reaches each point in the length of the beam the case may be considered as at "D." The bounding line to the maximum BM at every point is a parabola.

To find the BM diagram in "K," first suppose W_1 to be entirely removed. Then if $R'_2 =$ that part of R_2 which is due to W_2, $R'_2 = \frac{W_2 l}{p}$. Consider this force as acting upwards upon a beam supported at its two ends. Thus ABC is produced. Now consider only W_1, and draw ADB in the same way as is shown in "D," the maximum BM being set up vertically from AB. The BM area becomes as is shown shaded in the figure. Point E is a point of "contra-flexure," at which there is no bending moment.

The portion of W_1 carried to the left-hand end of beam $= \frac{nW_1}{p}$

Weight here required to oppose the turning effect of W_2 about $R_2 = \frac{oW_2}{p}$.

$$\therefore \text{Reaction } R_1 = \frac{nW_1}{p} - \frac{oW_2}{p}.$$

$$\text{and BM at } W_1 = \left(\frac{nW_1}{p} - \frac{oW_2}{p}\right)m.$$

R_2 may be found by subtracting this value of R_1 from $W_1 + W_2$. Length o of beam may be considered as a simple cantilever, and thus BM at $R_2 = W_2 \times o$. The length of beam between R_1 and E, the point of contra-flexure, may be considered as a simple beam like that shown in "D," the reaction at end E being supplied by a cantilever of length E to R_2. The BM caused by this reaction about R_2 must be equal to the BM caused by W_2 about R_2.

In "L" the method of procedure, as well as in the two following figures, should be apparent from what has gone before. pw may be taken as W_1, and ow as W_2.

In "M," supposing W_2 to be entirely removed, ABCD is drawn as in "G." Then considering W_2, BGC is drawn in the same manner as was ADB in "K." In this case there are two points of contra-flexure E and F.

"N" shows a case similar to the last, but with a distributed load.

At "O" and "P" beams are shown with both ends "fixed." Though there is a similarity between these two cases and those in "M" and "N," yet examples with fixed ends cannot be investigated in the same way, as, whatever the distribution of loading may be, the fixed end must always be in the same plane. The investigation which involves the deflection of the beam will not be entered upon here.[1] In the case shown at "O" the points of contra-flexure are found to be at a distance of $\frac{l}{4}$ from the supports. Between these points the beam may be considered as a simple beam supported at its ends and with a central load. The lengths of beam between the points of contra-flexure and the supports may be considered as simple cantilevers, with loads at their extremities equal to the reaction at the ends of the central beam, which $= \frac{W}{2}$.

In "P," with an evenly distributed load the points of contra-flexure are $.211l$ from the ends. The load on the cantilevers in this case = half the distributed load on the central beam acting at the extremity of the cantilever, the diagram for which is shown with a dotted line, besides the distributed load over the cantilever itself acting at half the length of the cantilever. The final curve forms a continuous parabola. The great advantage obtained by fixing the ends will be obvious on comparing the diagrams (see also values of m and n, page 67); but the results here shown depend upon the absolute fixing of the ends, and *absolute* fixing is very difficult to obtain. However, if the ends of a beam be only partially fixed there will be a corresponding increase in the relative strength of the beam; but it is impossible to determine the degree of fixity that will be attained when the beam is set in position. These considerations account for the fact that the apparent advantages of fixed ends are not generally relied upon, but it will be well, nevertheless, to bear those principles in mind. Another degree of uncertainty enters into the case, for if one abutment "settles" more than the other the beam will be initially strained, and the BM will be increased at the upper end of the beam.

[1] If more information upon this subject is required the reader is referred to *The Principles of Structural Design*, by Major Scott-Moncrieff.

Fig. 65 shows a simple method of describing a parabola. Its height is set up at CD, and EDF is drawn parallel to AB, and AE and BF are drawn perpendicularly to it. AE, ED, DF, and FB are each divided into an equal number of parts. Perpendicular lines are drawn through the points in EF, and lines are drawn from D to the points in AE and BF, points on

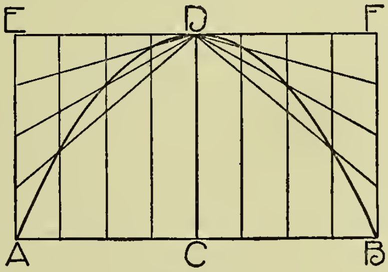


Fig. 65.

the required curve being given by the intersection of these lines.

Turning back to "L" in Fig. 64, it was here necessary to draw a curve cutting off vertical ordinates to the slanting line AB, of lengths equal to those of ordinates to a parabola on a horizontal base, as shown in a dotted line. This curve may be drawn at once as shown in Fig. 66, and as described above.

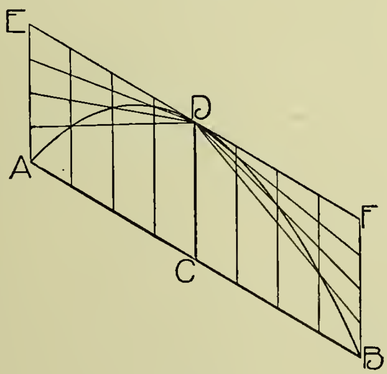


Fig. 66.

Stress Diagrams. — It has been shown that the moment of the stress in a beam = the bending moment at each point in the beam's length. Thus all the diagrams in Fig. 64 marked BM also show the moments of the stresses. Now, if in drawing the BM diagram the length of the beam be measured in terms of its effective depth (marked "*d*" in Figs. 50 to 56), the stress, tensional or compressional, at each point in the beam is given by the height of the curve at that point.

A useful method for drawing stress diagrams, known as Culmann's diagram, is shown in Fig. 67. The reactions R_1 and R_2 are found by the dotted polar and funicular diagrams as described in Chapter I. At *g*, the point dividing R_1 and R_2, *og* is set off at right angles to *af*, and of length *d* equal to the effective depth of the beam (see Figs. 50 to 56). The funicular polygon KNP now obtained with the aid of the new pole *o* is the required stress diagram.[1] The closing of the polygon KNP

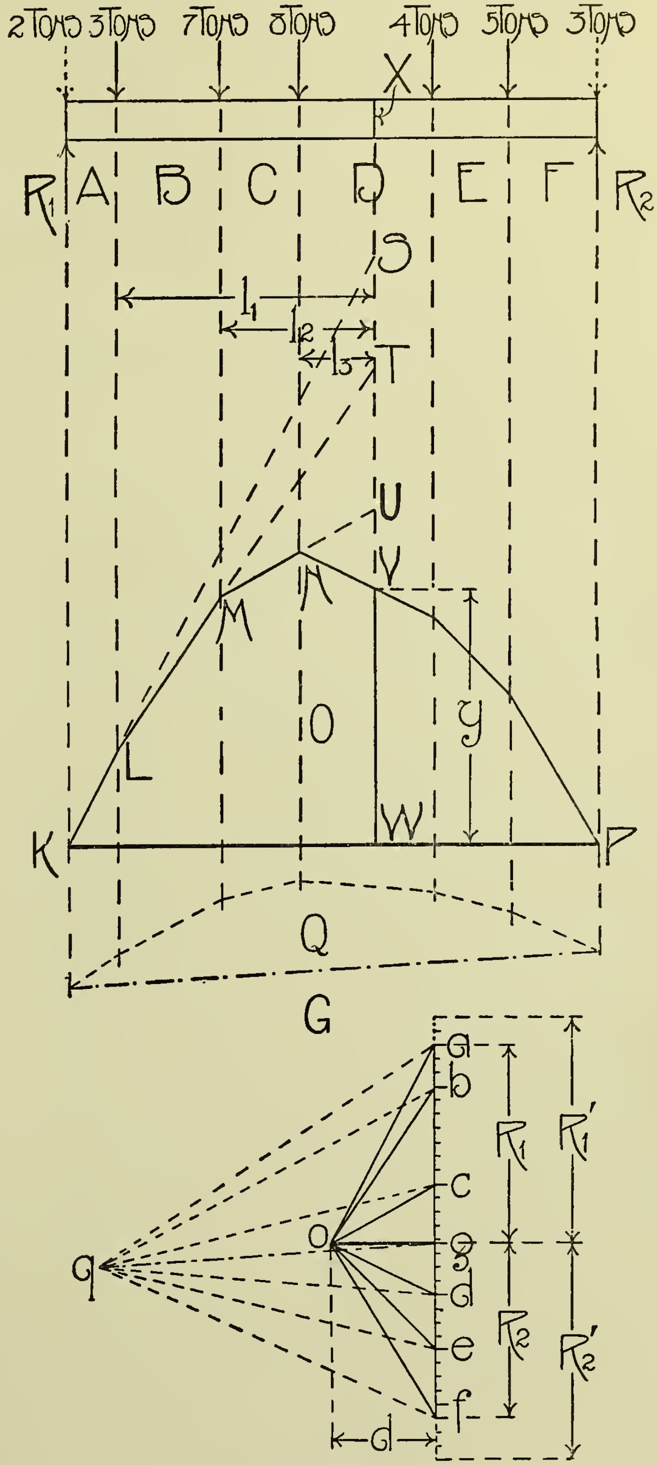


Fig. 67.

will depend upon the accuracy of drawing. Consider the moments about any point X. In the two diagrams

[1] In order to reduce the height of the funicular polygon, *og* may be made a multiple of *d*, say 3*d*; but in this case the ordinates to the funicular polygon, measured upon the diagram, must likewise be multiplied by 3 to find the true stresses in the beam.

the triangles KSW and *oag* are similar. $\therefore \frac{SW}{KW}=\frac{ag}{d}$.

$\therefore$ SW $\times d = ag \times$ KW, which = moment of R_1 about X.

The triangles LST and *oab* being similar, $\frac{ST}{l_1}=\frac{ab}{d}$.

$\therefore$ ST $\times d = ab \times l_1$, which = moment of weight of 3 tons.
Similarly TU $\times d = bc \times l_2 =$,, ,, 7 ,,
,, UV $\times d = cd \times l_3 =$,, ,, 8 ,,

Bending moment at X = moment of R_1 − moments of the weights 3, 7, and 8 tons.

$\therefore$ BM at X = SW $\times d$ − ST $\times d$ − TU $\times d$ − UV $\times d$.
= (SW − ST − TU − UV)$d = y \times d$.
$\therefore$ Moment of stress at X $= y \times d$.

$\therefore$ Total stress at X, either in compression or tension, $= y$, being measured on the same scale to which af is set down.

It should be noted that the dotted polygon found with pole q is also a BM diagram.

The weight shown dotted at the two extremities of the beam do not affect the BM, but merely increase the reaction, as R'_1 and R'_2.

The stresses produced in the simplest cases of loaded beams are given by the following formulæ: where f= stress, d= effective depth, l= length of beam, and W = load, then in a cantilever with end load, $f \times d = W \times l$.

$$\therefore f = \frac{Wl}{d}.$$

Similarly the following—

Cantilever with distributed load, $f=\frac{Wl}{2d}$.

Beam with central load, . . $f=\frac{Wl}{4d}$.

Beam with distributed load, . $f=\frac{Wl}{8d}$.

Beams of Varying Depths.—It has been shown above that ordinates to the BM curves in Fig. 64 are equal to $f \times d$, where f = stress in tension or compression, and d= effective depth of beam. It has been shown also that if the depth of the beam be constant, the stress in either half of the beams section is represented by the ordinates. Similarly, if the stresses in the flanges of the beam are to remain constant, the depth of the beam must vary, and the curve may now be taken to represent the outline of the beam. That is to say, if girders were constructed having the outline given at BM in Fig. 64 and similarly loaded, the stresses in the flanges would be constant throughout the length.

Effective Span.—A girder or joist will have a bearing of a certain length throughout which the reaction

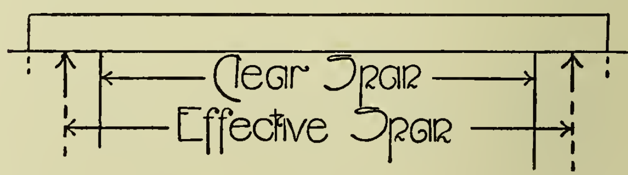


Fig. 68.

of the abutment is imparted to it; but for sake of calculation the reaction is supposed to act at the central point of the bearing. The effective span (see Fig. 68) is assumed to be the distance between the central points of the bearings, so that effective span = clear span + length of *one* bearing. In all the illustrations given above, the length of the beams represented is the "effective" length.

CHAPTER IV

BEAMS FURTHER CONSIDERED

STRESS-STRAIN DIAGRAMS. — Fig. 69 is a stress-strain diagram showing the characteristic behaviour of a rod of steel when gradually loaded in tension until it breaks at point D. The load is set up along OY, and the corresponding extension of bar is measured along OX. The curve OABCD passes through the intersection of ordinates from corresponding points on OY and OX. The curve from O to EL forms a straight line,—or in other words, throughout this interval strain is proportioned to stress. When strained up to this point the rod will resume its original form on the removal of the load. This point EL is known as the "Elastic Limit" of the metal. Beyond EL the metal becomes partially

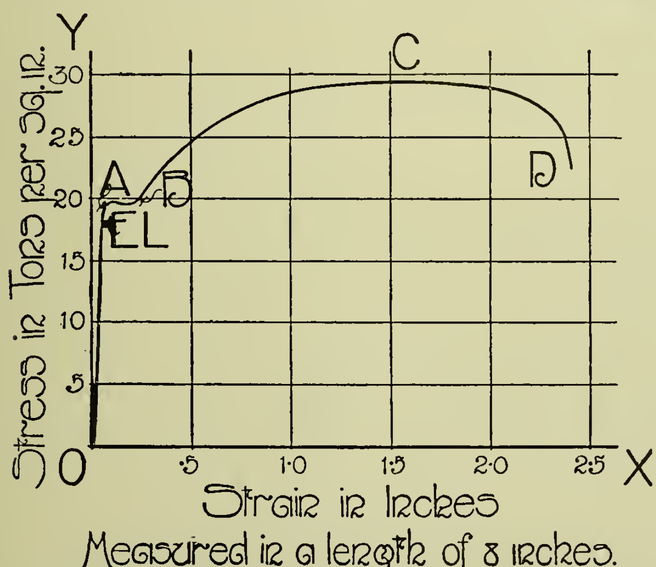


FIG. 69.

plastic, and "breaks down" at A, but partially recovers itself at B. The fall of the curve from C and D is due to the fact that the section of the rod is decreased so that a smaller load gives an equal stress per square inch of original cross section. The stress set down on line OY is the stress per square inch of *original* section. Metal in structures must never be strained beyond the elastic limit. The load necessary to strain steel or wrought iron to its elastic limit is generally about 65 per cent. of the breaking load.

MODULUS OF ELASTICITY, or YOUNG'S MODULUS, is a number giving the ratio of stress and strain within the elastic limit of any particular material. This number, which is always denoted by the letter "E," is practically the same in compression and tension.

E = stress per square inch in lbs. or tons ÷ strain per inch of length $=\frac{f.\,l}{x}$,—where f = stress per square inch, l = length of test piece in inches, and x = strain or extension in inches. E is thus equal to an ideal load per square inch which will double the length of the material elastically; for when $x=l$, then $E=f$.

DEFLECTION OF BEAMS.—Any load placed upon a beam will produce a corresponding deflection in the beam. If a beam be employed to support a brick wall or a plaster ceiling, any undue deflection in the beam will cause unsightly cracks. It is very important, then, to design beams in such cases with due regard to their stiffness, and this consideration will often necessitate the use of more metal in a girder than is actually necessary for its strength. The maximum practical allowance for deflection is generally taken at $\frac{1}{40}$ inch per foot of span, but this may sometimes be increased to $\frac{1}{30}$ inch. The outer layer of a beam will be strained at each point in its length by an amount proportional to the maximum fibre stress at that point. Thus the total length of this outer layer will be reduced on the compression side and correspondingly increased on the tension side of the beam; and from the relative length of these two layers the deflection of the beam may be calculated. Space will not permit the calculations necessary to arrive at a general formula being given here, but the results are as follows (see page 105).

$$D = m \cdot \frac{Wl^3}{EI} = n \cdot \frac{fl^2}{Ey}.$$

D = maximum deflection in inches.
m and n = factors given in the table below.
W = total load, concentrated or distributed.
l = length of beam in inches.
E = modulus of elasticity.
I = moment of inertia in inches.
f = maximum fibre stress. (If this is different in the two flanges the mean may be taken.)
y = distance of outer fibre from neutral axis in inches.

Note.—W, E, and f must be of the same denomination.

Bending Moment Diagrams for Continuous Girders

A. Two Spans
B. Three ,,
C. Four ,,
D. Five ,,
E. Six ,,

Fig. 70.

Values of m and n.

	Uniform Cross Section.		Uniform Depth. Flange Area proportional to BM.	
	m	n	m	n
Cantilever, concentrated load	$\frac{1}{3}$	$\frac{1}{3}$	$\frac{1}{2}$	$\frac{1}{2}$
Cantilever, distributed load	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{2}$
Beam, supported, central load	$\frac{1}{48}$	$\frac{1}{12}$	$\frac{1}{32}$	$\frac{1}{8}$
Beam, supported, distributed load . . .	$\frac{5}{384}$	$\frac{5}{48}$	$\frac{1}{64}$	$\frac{1}{8}$
Beam, fixed ends, central load	$\frac{1}{192}$	$\frac{1}{24}$	$\frac{1}{128}$	$\frac{1}{16}$
Beam, fixed ends, distributed load . .	$\frac{1}{384}$	$\frac{1}{16}$*	$\frac{1}{384}$†	$\frac{1}{16}$

* $\frac{1}{32}$ if value of f at abutments instead of at centre is used in the formula.

† $\frac{1}{192}$ if value of I at abutments instead of at centre is used in the formula.

The values of m and n in the first and second columns of the above table are used when calculating the deflection of a beam of uniform section, such as a rolled steel joist, and the values in the third and fourth columns are used in cases where the amount of metal at every point in the length of the beam is so proportioned that the intensity of stress is constant throughout its length. A plate girder is an example of the latter case; however, the flange area is not here perfectly proportioned to the BM at each point in the length of the beam, and consequently the deflection, theoretically, would have a value somewhere between the values arrived at by the use of first one and then the other of the above formulæ. However, the fact that the section is not homogeneous will tend to increase the deflection. Thus the second values of m and n will give the nearest approximation to accuracy. In the second case, too, no matter what be the distribution of loading, so long as the uniform intensity of stress be known, the deflection may be calculated from the formula $D = n \cdot \frac{fl^2}{Ey}$, where the value of $n = \frac{1}{2}$ in the case of a cantilever, $\frac{1}{8}$ in the case of a supported beam, and $\frac{1}{16}$ in the case of a fixed beam.

Continuous Beams.—In Fig. 64 the case of a single beam with more than two supports was not considered. The case is similar to that of the fixed beam, with the exception that such parts of the beam as occur directly over the supports are not held rigidly in a truly horizontal plane.

Fig. 71 shows a continuous beam uniformly loaded over two equal spans. Imagine the central support to be removed. The beam will then sag as shown dotted, and the amount of the deflection at the centre $= \frac{5}{384} \cdot \frac{Wl^3}{EI}$ (see last paragraph). Now, in order to bring back the beam to its original position at the centre a force must be applied at this point of an amount which would cause the same deflection upwards that the distributed load caused downwards. Therefore if this force $= W_1$,

$$\text{Then } \frac{5}{384} \cdot \frac{Wl^3}{EI} = \frac{1}{48}\frac{W_1 l^3}{EI}.$$

$$\therefore W_1 = \frac{5}{8} W.$$

Thus the reaction at the central support $= \frac{5}{8}$ of the total load on the beam, or $\frac{10}{8}$ of the load over one span.

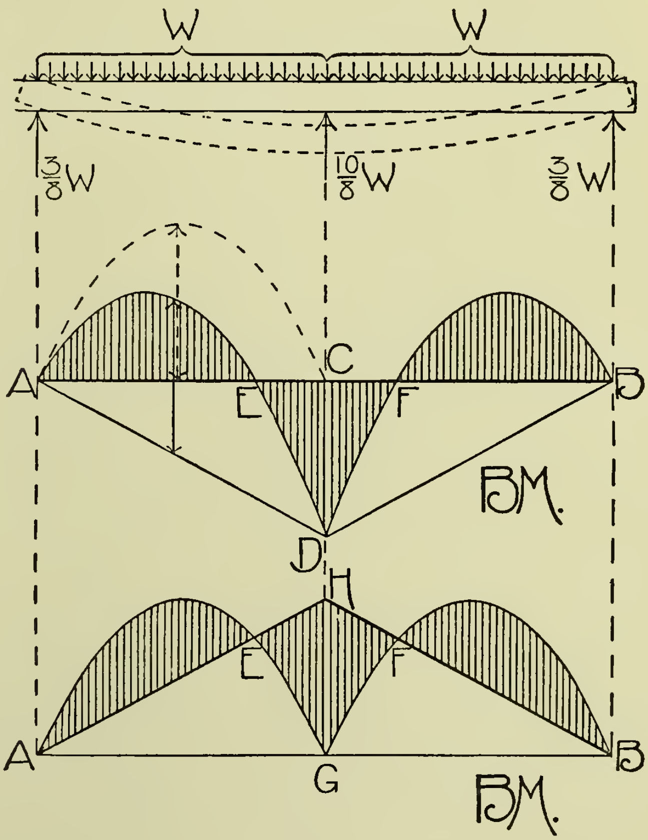


Fig. 71.

The rest of the load is divided between the two end supports.

Fig. 70 shows the BM diagrams and reactions of the supports for continuous beams with evenly distributed loads and with from two to six equal spans. These data have been calculated from Clapeyron's "Theorem of Three Moments," which is as follows—

For uniformly distributed loads—

$$M_1 l_1 + 2M_2(l_1 + l_2) + M_3 l_2 = \tfrac{1}{4}(w_1 l_1^3 + w_2 l_2^3).$$

For concentrated loads—

$$\frac{M_1 l_1}{6} + \frac{M_2}{3}(l_1 + l_2) + \frac{M_3 l_2}{6} = \frac{\Sigma W_1 m_1}{6 l_1}(l_1^2 - m_1^2) + \frac{\Sigma W_2 m_2}{6 l_2}(l_2^2 - m_2^2).$$

M_1, M_2, and M_3 = bending moments at three consecutive supports.

l_1 and l_2 . . = lengths of the two spans.

w_1 and w_2 . . = distributed load per foot run over the two spans.

W_1 and W_2 . = concentrated loads on the two spans.

m_1 and m_2 . . = distance of these concentrated loads from the outer extremities of the two spans under consideration.

$\Sigma W_1 m_1$ . . . = the sum of the moments of all loads on l_1. If W_1', W_1'', W_1''', etc. be loads on l_1, at distances of m_1', m_1'', m_1''', etc. from the outer extremity of l_1, then $\Sigma W_1 m_1 = W_1' m_1' + W_1'' m_1'' + W_1''' m_1''' +$ etc.

Referring again to Fig. 71, having found by the above theorem that the BM over the central support $= \frac{Wl}{8}$, set down a line CD representing this amount on the diagram, join AD and DB, and on these lines draw curves giving the BM as for a separately supported beam over each span. (The method of drawing these curves is shown in Fig. 66.) The shaded area gives the final BM diagram, with points of contra-flexure at E and F. Instead of drawing CD below the line AB, it may be drawn above it as GH, the parabolas for each span being drawn in the ordinary way. The base line now takes the broken form AHB. The latter method is simpler, especially where several spans are involved, as shown in Fig. 70. The shear diagram for a 6-span beam is also shown in Fig. 70, and should appear obvious to those who have properly grasped the principles illustrated on Fig. 64.

By way of an example to illustrate the application of the Theorem of Three Moments, a beam of three unequal spans is assumed to be loaded as shown in Fig. 72. First consider spans AB and BC; then by the second formula, if M_a, M_b, M_c, and M_d = bending moments at A, B, C, and D respectively—

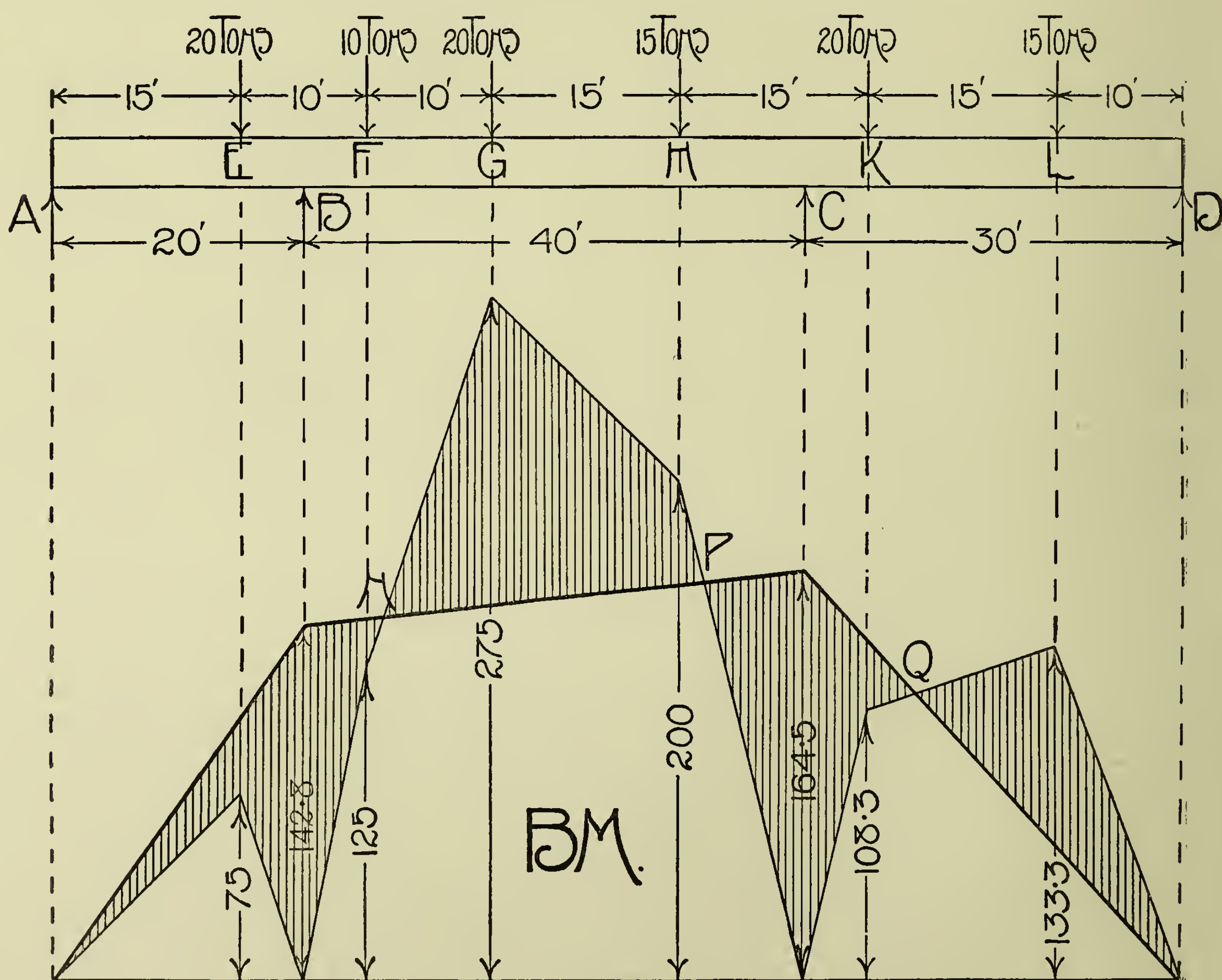


Fig. 72.

$$\frac{Ma \times AB}{6} + \frac{Mb}{3}(AB + BC) + \frac{Mc \times BC}{6}$$

$$= \left[\frac{20 \times AE}{6\,AB}(AB^2 - AE^2)\right] + \left[\frac{10 \times FC}{6 \times BC}(BC^2 - FC^2)\right.$$

$$\left. + \frac{20 \times GC}{6 \times BC}(BC^2 - GC^2) + \frac{15 \times HC}{6 \times BC}(BC^2 - HC^2)\right].$$

$Ma = 0.$

$$\therefore \frac{Mb \times 60}{3} + \frac{Mc \times 40}{6} = \frac{20 \times 15}{6 \times 20}(400 - 225)$$

$$+ \frac{10 \times 35}{6 \times 40}(1600 - 1225) + \frac{20 \times 25}{6 \times 40}(1600 - 625)$$

$$+ \frac{15 \times 10}{6 \times 40}(1600 - 100).$$

Simplifying this equation—

$$3\,Mb + Mc = 593. \quad \text{(i)}$$

Now consider spans BC and CD—

$$\frac{Mb \times BC}{6} + \frac{Mc}{3}(BC + CD) + \frac{Md \times CD}{6}$$

$$= \left[\frac{10 \times BF}{6 \times BC}(BC^2 - BF^2) + \frac{20 \times BG}{6 \times BC}(BC^2 - BG^2)\right.$$

$$\left. + \frac{15 \times BH}{6 \times BC}(BC^2 - BH^2)\right] + \left[\frac{20 \times KD}{6 \times CD}(CD^2 - KD^2)\right.$$

$$\left. + \frac{15 \times LD}{6 \times CD}(CD^2 - LD^2)\right].$$

$Md = 0.$

$$\therefore \frac{Mb \times 40}{6} + \frac{Mc \times 70}{3} = \frac{10 \times 5}{6 \times 40}(1600 - 25)$$

$$+ \frac{20 \times 15}{6 \times 40}(1600 - 225) + \frac{15 \times 30}{6 \times 40}(1600 - 900)$$

$$+ \frac{20 \times 25}{6 \times 30}(900 - 625) + \frac{15 \times 10}{6 \times 30}(900 - 100).$$

Simplifying this equation—

$$Mc = \frac{68975}{336} - \frac{2Mb}{7}.$$

Substituting in (i)—

$$Mb = 142.8.$$

$$\text{and } Mc = 164.5.$$

The bending moments over the supports now being known, the BM diagram is drawn as described in the last paragraph, and as shown in Fig. 72. Points of contra-flexure are shown at N, P, and Q. It should be noticed that there is no point of contra-flexure in span AB, and that the upper part of beam is here all in tension, while the lower part is all in compression ; that is, the deflection of this part of the beam takes place in an upward direction.

The calculation for a continuous beam as shown above is seen to be an arduous task. An entirely graphic method is given by Professor Claxton Fidler in his book on Bridge Construction.

The settlement of a support will largely alter the distribution of stress in a continuous girder, and this fact has led to mistrust of the data given by theoretic computation. However, the principle may be used with advantage, especially with evenly distributed loads and large spans.

Factor of Safety.—It was shown at the commencement of this chapter that if metal be strained beyond its elastic limit a permanent injury is done to it, and as this point is reached by the application of little more than half the breaking load it is evident that the working load must never be greater than half the breaking or ultimate load. Thus a "factor of safety" of two is introduced. Again, we cannot be sure that all the metal will be equal to the tested specimen, and that there are no flaws in the sections used, besides which the structure may be weakened by imperfect workmanship, and in course of time by rust. To guard against the latter defects it is usual to halve the maximum stress that may be put upon the metal, and consequently the factor of safety becomes equal to four. For instance, if it is found that certain metal will stand an ultimate load of 28 tons to the square inch, the greatest load that will be brought upon the metal in a structure should be 7 tons. It is seen that the factor of safety depends upon the ratio of the elastic limit stress, or "proof stress," to the ultimate stress, and also upon the liability of the particular metal to flaws. Thus in dealing with steel or wrought iron a factor of 3 or 4 may be used, whilst in the case of cast iron it would be better to use a factor of 5 or 6.

Live Loads.—If a force equal to the weight of 1 lb. move a body through a distance of 1 foot, the "Work" done = 1 lb. × 1 foot = 1 foot-lb. Now, if a load of W tons be applied to a bar, producing an extension of x inches, the work done on the bar = Wx ton-inches. If, again, the same load W be applied *gradually*, the final extension will be x inches, but the average distance through which all portions of W have acted = $\frac{x}{2}$. Thus in this case the work done on the bar = $\frac{Wx}{2}$ = half the work done by the load if suddenly applied. The stress set up in the bar when the load is suddenly applied is therefore twice that set up by the gradually applied load. The stress caused by the load is ultimately the same in both cases, but in the first case a number of oscillations are set up.

These facts are born out by Wöhler's and Bauschinger's experiments, which show that a bar may be broken by the repeated action of a load which is only slightly in excess of the elastic limit, if only the load be applied a sufficient number of times.

According to the above facts, when dealing with "live loads" the factor of safety should be doubled ; but few loads, if any, will come upon a structure with the suddenness indicated above, so that it should be sufficient to multiply the factor of safety by $1\frac{1}{2}$.

All permanent loads on a structure are known as "dead loads" ; all other loads, moving or capable of being moved, are considered as "live loads."

Instead of altering the factor of safety for dead and live loads, it is usual and more convenient to consider the total load upon a structure as equal to the dead load + twice the live load.

The Properties of Iron.—The data given in the

table at foot of page are average values, and it must be borne in mind that the actual values may vary to a considerable extent on either side of those in the table. To obtain accurately the values for any particular case, tests must be applied to samples made in the same way and at the same time as the metal in question, and even then the results from two samples may differ widely. This uncertainty is, however, covered by the factor of safety.

The ultimate compressive resistance for wrought iron and steel is placed in brackets in the table printed below, as it cannot be accurately measured on account of the ductility of the materials.

Steel may be obtained of very much greater strength than that indicated in the table, but the softer qualities of "mild steel" are generally considered preferable, being more easily worked and more reliable in working.

	Weight in Lbs. per Ft. super. 1 Inch thick.	Ultimate Resistance.			Safe Resistance.			Modulus of Elasticity (E). Units, Inches, and Tons.
		Tension.	Compression.	Shear.	Tension.	Compression.	Shear.	
Cast iron . .	37.5	7	36	5	1½	7	1	7,500
Wrought iron .	40.4	22	(18)	20	5	4	4	12,000
Steel . . .	40.8	28	(28)	22	6½	6½	5	13,500

Table of Approximate Safe Loads upon Bearing Surfaces.

Material.	Safe Load. Tons per Sq. Ft.	Material.	Safe Load. Tons per Sq. Ft.
Granite	20 to 30	London stocks in cement mortar, 1–4	8
Building stone	12 to 20	London stocks in Lias lime mortar	5
Staff. blue brick in cement mortar, 1–4	15	London stocks in ordinary lime mortar	3
Flettons in cement mortar, 1–4	10	Cement concrete, average quality 1 to 6, after 1 month	2½
Flettons in Lias lime mortar	7	Cement concrete, average quality 1 to 6, after 6 months	10
Flettons in ordinary lime mortar	5	Cement concrete, average quality, 1 to 6, after 12 months	15

CHAPTER V

ROLLED STEEL JOISTS

The Engineering Standards Committee is a body representing all the most important engineering institutions of England. It is engaged in drawing up standard dimensions for all common objects used in every branch of the profession. In this way standard sections for rolled joists, angles, channels, etc. have been drawn up, and the various properties of each have been calculated and published. It is usual to specify a rolled joist by its outside dimensions and its weight per foot run, such as 15×6 inches, 59 lbs. per foot run (see table below); but these quantities give no hint as to the distribution of the metal, and it is upon the distribution that the strength of the beam depends. Many makers have now adopted the sections recommended by the Committee, others have not, while others again roll special sections as well as the standard sections of the Committee.

TABLE I.—PROPERTIES OF SECTIONS.

(Published by Permission of the Engineering Standards Committee.)

Reference No.	Size. Inches.	Weight per Foot. Lbs.	Area. Sq. Ins.	Diagram. Web t	Diagram. Flange T	Diagram. Radius R1	Diagram. Radius R2	Moment of Inertia. about x—x	Moment of Inertia. about y—y	Radius of Gyration. Inches. about x—x	Radius of Gyration. Inches. about y—y	Resistance about x—x Inch. Sq. Inches.	Cross Centres of Holes C. Inches.
B.S.B. 30	24×7½	100	29.392	.6	1.07	.7	.35	2654.769	66.874	9.504	1.508	221.231	4.5
,, 29	20×7½	89	26.164	.6	1.01	.7	.35	1671.291	62.586	7.992	1.547	167.129	4.5
,, 28	18×7	75	22.066	.55	.928	.65	.325	1149.667	46.618	7.218	1.453	127.741	4
,, 27	16×6	62	18.227	.55	.847	.65	.325	725.953	27.069	6.311	1.219	90.744	3.5
,, 26	15×6	59	17.346	.5	.88	.6	.3	629.094	28.203	6.022	1.275	83.879	3.5
,, 25	15×5	42	12.351	.42	.647	.52	.26	428.207	11.937	5.888	.983	57.094	2.75
,, 24	14×6	57	16.769	.5	.873	.6	.3	533.091	27.941	5.638	1.291	76.156	3.5
,, 23	14×6	46	13.533	.4	.698	.5	.25	440.625	21.584	5.706	1.263	62.946	3.5
,, 22	12×6	54	15.879	.5	.883	.6	.3	375.599	28.280	4.863	1.334	62.560	3.5
,, 21	12×6	44	12.946	.4	.717	.5	.25	315.439	22.257	4.936	1.311	52.573	3.5
,, 20	12×5	32	9.408	.35	.55	.45	.225	220.115	9.743	4.837	1.018	36.686	2.75
,, 19	10×8	70	20.582	.6	.97	.7	.35	345.039	71.609	4.094	1.865	68.008	4.75
,, 18	10×6	42	12.358	.4	.736	.5	.25	211.614	22.930	4.138	1.362	42.323	3.5
,, 17	10×5	30	8.820	.36	.552	.46	.23	145.684	9.780	4.064	1.053	29.137	2.75
,, 16	9×7	58	17.064	.55	.924	.65	.325	229.740	46.265	3.669	1.647	51.053	4
,, 15	9×4	21	6.178	.3	.46	.4	.2	81.115	4.198	3.624	.824	18.026	2.25
,, 14	8×6	35	10.293	.44	.597	.54	.27	110.597	17.929	3.278	1.320	27.649	3.5
,, 13	8×5	28	8.241	.35	.575	.45	.225	89.357	10.250	3.293	1.115	22.339	2.75
,, 12	8×4	18	5.297	.28	.402	.38	.19	55.716	3.574	3.243	.821	13.929	2.25
,, 11	7×4	16	4.709	.25	.387	.35	.175	39.222	3.410	2.886	.851	11.206	2.25
,, 10	6×5	25	7.354	.41	.52	.51	.255	43.641	9.105	2.436	1.113	14.547	2.75
,, 9	6×4½	20	5.882	.37	.431	.47	.235	34.660	5.409	2.427	.959	11.553	2.5
,, 8	6×3	12	3.527	.26	.348	.36	.18	20.228	1.338	2.395	.616	6.743	1.5
,, 7	5×4½	18	5.290	.29	.448	.39	.195	22.699	5.656	2.071	1.034	9.080	2.5
,, 6	5×3	11	3.238	.22	.376	.32	.16	13.620	1.461	2.051	.672	5.448	1.5
,, 5	4¾×1¾	6.5	1.912	.18	.325	.28	.14	6.767	.263	1.881	.371	2.849	
,, 4	4×3	9.5	2.795	.22	.336	.32	.16	7.526	1.280	1.641	.677	3.763	1.5
,, 3	4×1¾	5	1.472	.17	.24	.27	.135	33.671	.194	1.579	.363	1.835	
,, 2	3×3	8.5	2.501	.2	.332	.3	.15	3.789	1.261	1.231	.710	2.526	1.5
,, 1	3×1½	4	1.176	.16	.248	.26	.13	1.657	.124	1.187	.325	1.105	

As an example of special sections may be mentioned the broad flanged beams, or "Differdange Beams," rolled in England by Messrs. Skelton & Co. By the

use of the Grey Mill they are enabled to roll sections up to $11\frac{7}{8}$ inches in width, the largest section they roll being $29\frac{1}{2} \times 11\frac{7}{8}$ inches, weighing 177 lbs. per foot run. The broad flange is particularly useful in giving a well-proportioned section in the large sizes.

Makers' Tables.—All makers of rolled joists publish tables of safe distributed loads over various spans for the necessary section to carry a given distributed load over a given span. The loads given here are calculated to a maximum fibre stress of $7\frac{1}{2}$ tons to the square inch. The maximum safe stress allowed for in different makers' lists differs widely. This is chiefly due to their taking a different factor of safety, as well, perhaps, as to variation in the quality of the metal.

TABLE II.—SAFE DISTRIBUTED LOADS IN TONS ON BEAMS OF VARYING SPANS.

(From Messrs. Dorman, Long, & Co.'s List.)

	LENGTH OF SPAN IN FEET.																									
Reference No.	2	4	6	8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50	Deflection Coefficient.
B. S. B. 30					102	92	79	69	61	55	50	46	42	39	36	34	32	30	29	27	26	25	24	23	22	.00078
,, 29				94	83	69	59	52	46	41	38	34	32	29	27	26	24	23	22	20						.000937
,, 28				78	64	53	45	40	35	32	29	26	24	22	21	20	18	17	16							.00104
,, 27			73	56	45	38	32	28	25	22	20	19	17	16	15	14	13	12	11							.00117
,, 26			62	52	42	35	30	26	23	21	19	17	16	15	14	13	12	11								.00125
,, 25			47	35	28	24	20	18	16	14	13	12	11	10	9.5	9	8.4									.00125
,, 24			59	47	38	31	27	24	21	19	17	16	14	13	12											.00133
,, 23			43	39	31	26	22	19	17	15	14	13	12	11	10											.00133
,, 22		53	52	39	31	26	22	19	17	15	14	13	12	11	10											.00156
,, 21			40	33	26	22	19	16	14	13	12	11	10	9												.00156
,, 20		32	30	23	18	15	13	11	10	9	8.3	7.6	7	6.5												.00156
,, 19			53	43	34	28	24	21	19	17	15	14	13													.001875
,, 18			35	26	21	17	15	13	11	10	9.6	8.8	8													.001875
,, 17		30	24	18	14	12	10	9	8	7.2	6.6	6	5.6													.001875
,, 16		44	42	32	25	21	18	16	14	12	11	10	9.8													.00208
,, 15		22	15	11	9	7.5	6.4	5.6	5	4.5	4	3.7														.00208
,, 14		31	23	17	14	11	9.8	8.6	7.7	7	6.3															.00234
,, 13		25	18	14	11	9	8	7	6.2	5.5	5															.00234
,, 12	19	17	11	8.7	7	5.8	5	4.3	3.8	3.5	3.2															.00234
,, 11	15	14	9.4	7	5.6	4.7	4	3.5	3.1	2.8	2.5															.00268
,, 10	22	18	12	9	7.3	6	5.2	4.5	4																	.003125
,, 9	20	14	9.6	7.2	5.8	4.8	4.1	3.6	3.2																	.003125
,, 8	14	8.4	5.6	4.2	3.4	2.8	2.4	2.1	1.9																	.003125
,, 7	13	11.3	7.6	5.6	4.5	3.8	3.2	2.8																		.00375
,, 6	9.8	6.8	4.5	3.4	2.7	2.3	1.9	1.7																		.00375
,, 5	7	3.5	2.4	1.8	1.4	1.2	1																			.00395
,, 4	7.8	4.7	3.1	2.3	1.9	1.6	1.3																			.00469
,, 3	4.6	2.3	1.5	1.1	.91	.76	.65																			.00469
,, 2	5.3	3.2	2	1.6	1.2	1	.9																			.00625
,, 1	2.8	1.4	·92	.6	.55	.46	.39																			.00625

The loads given in the table are based on an extreme fibre stress of $7\frac{1}{2}$ tons per square inch, being one-fourth of the average breaking stress, and are calculated on the assumption that the beams receive the usual side support as in building work. For other cases, such as concentrated or live loads, special calculation is necessary.

Care should be taken in selecting beams that the load is not so great as to cause excessive deflection, which may occur if the span exceeds 20 times the depth. This limit is indicated by the zigzag line in the table.

The deflection (in inches) for tabular load is found by multiplying the square of the span in feet by the coefficient which is given for each section. If the actual load is less than the tabular load the deflection will be less in exactly the same proportion.

The customary margin required in rolling is $2\frac{1}{2}$ per cent. over or under the specified weights, and it is rare for a guarantee to be given to roll sections without this allowance.

As a rule all beams, either from rolls or stock, are cut to a margin of 1 inch over or under specified lengths; and an extra is charged for cutting beams to within an eighth of an inch of exact length and for machining square.

each particular section, though the data on which these values are calculated are given less fully by some makers than by others. Table I. is taken from the Properties of British Standard Sections, published by the Engineering Standards Committee and adopted by many manufacturers, in which the dimensions are given fully. Table II.—Safe distributed loads. This is taken from Messrs. Dorman, Long, & Co.'s list. It is seen that it is only necessary to glance at the table to find the necessary section to carry a given distributed load over a given span. For ordinary cases it will be sufficient to choose the required section from the list of a reliable firm, but where a large quantity is required it will be well to satisfy oneself as to the ultimate strength of the metal, and to employ one's own factor of safety.

Rolled steel joists may be obtained of lengths up to 50 feet and 3 to $29\frac{1}{2}$ inches in depth. They may generally be used with economy for all loads of moderate proportions, but the larger sizes may often

preferably be substituted by plate girders; for there is of necessity a large amount of surplus metal at the extremities of an R.S.J., and where it is necessary to carry only a small load over a large span a

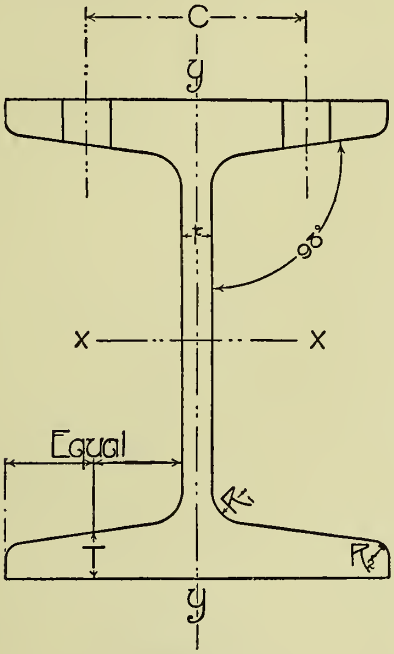


FIG. 73.

light lattice girder may probably with advantage take the place of a rolled steel joist.

THE SELECTION OF A ROLLED JOIST.

By way of example, select a rolled steel joist to carry a 9-inch brick wall 11 feet high over a 20-feet clear span.

We may take the weight of stock brickwork as roughly 1 cwt. per cubic foot.

∴ Load per foot run $=\frac{3}{4}\times 1\times 11=8\frac{1}{4}$ cwts.

A beam of uniform section must be designed to meet the greatest stresses that will be brought upon it at any point in its length.

It may be assumed that the joist has a bearing of 1 foot length upon each abutment.

The effective span is now 21 feet, and the effective load $=21\times 8\frac{1}{4}=8$ tons, 13 cwts.

Referring to the table, it is seen that a joist 12×5 inches weighing 32 lbs. per foot run will bear a distributed load of 9 tons over a span of 20 feet and a load of 8.3 tons over a span of 22 feet. This joist will evidently be just strong enough to carry the required load.

The deflection coefficient for this section is shown to be .00156.

∴ The total deflection $=.00156\times 21^2=.7$ inch. (See note at foot of table. The Deflection Coefficient $=n\cdot\frac{f}{Ey}$ in the formula already given. The coefficient may be found by experiment for any section and material.) This deflection, which is equivalent to $\frac{1}{33}$ inch per foot run, might well be considered to be excessive for the purpose under consideration, so that a slightly heavier section should be employed, say 12×6 inches, weighing 44 lbs. per foot run.

The lighter section has the further disadvantage that it lacks lateral stiffness unless it be supported within the span in this direction. For beams which are unsupported laterally the width of flanges should not be less than $\frac{1}{40}$ of span.

For the sake of demonstration it will be well now to go through the calculations to find the strength and deflection of this joist, 12×6 inches and 44 lbs. per foot run.

THE SUPPORT OF THE JOIST.—The total load over the clear span $=8\frac{1}{4}\times 20$ cwts. $=8\frac{1}{4}$ tons.

$$\text{Weight carried by each abutment}=\frac{8\frac{1}{4}}{2}=4\frac{1}{8}\text{ tons.}$$

The actual load upon the abutments will be somewhat increased by the wall immediately above. If desired this may be allowed for.

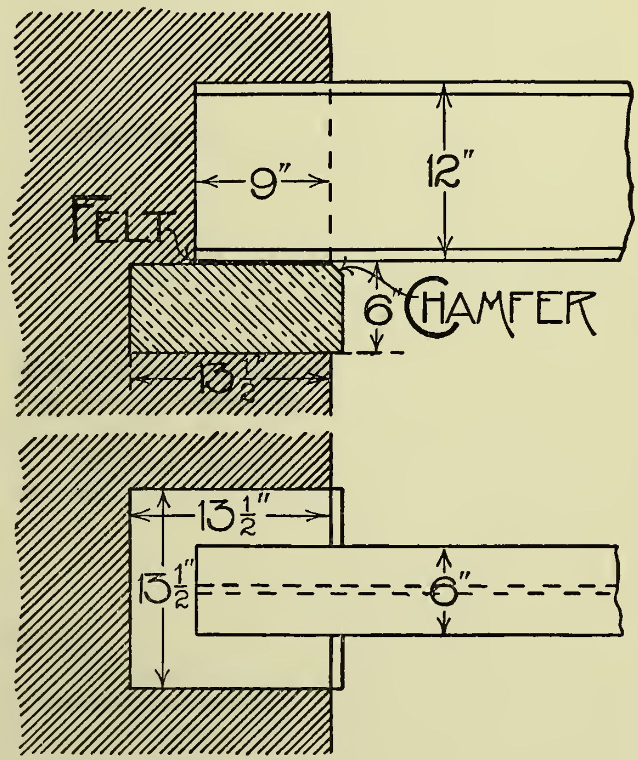


FIG. 74.

The ends of the joist will rest upon pads of stone upon which a safe load of 12 tons per square foot may be allowed.

∴ As the width of the joist = 6 inches, the length of its bearing must be $\frac{4\frac{1}{8}\times 144}{12\times 6}=8\frac{1}{4}$ inches, say 9 inches (see Fig. 74).

If, in certain cases, the necessary length of bearing becomes unduly long, a cast-iron plate may be inserted

between the joist and the stone, thus obtaining a wider bearing and enabling its length to be reduced.

The safe load upon brickwork in lime mortar may be taken as 4 tons per foot super.

$\therefore$ The necessary bearing area upon the brickwork is three times that upon the stone.

$\therefore$ The stone pad must have an area of 6×9 inches $\times 3 = 162$ square inches.

It must have brick dimensions, and has here been made $13\frac{1}{2} \times 13\frac{1}{2}$ inches.

In order to prevent the edge of the stone from chipping it should be chamfered. It will be well to insert a piece of felt between the iron and the stone to obtain a more even bedding.

The 9-inch wall may be built directly upon the 6-inch flange. With a thicker wall stone slabs would be bedded upon the joist to receive the wall, or an iron plate may be riveted to the upper flange of the necessary width.

STRENGTH.—The maximum BM $= \frac{Wl}{8}$; and the moment of resistance $= \frac{I}{y} \cdot f$.

These two turning effects must equate.

$$\therefore \frac{Wl}{8} = \frac{I}{y} f.$$

$l =$ effective span $= 20$ feet 9 inches $= 249$ inches.

I, according to the table $= 315.3$.

$y = \frac{d}{2} = 6$ inches.

f may be taken as $7\frac{1}{2}$ tons per square inch.

Thus the equation becomes—

$$\frac{W \times 249}{8} = \frac{315.3 \times 15}{6 \times 2}.$$

$\therefore$ W $= 12\frac{1}{2}$ tons approximately $=$ the distributed load that the beam is capable of supporting with safety. The actual load to be supported by the joist over the effective span $= 8\frac{1}{4}$ cwts. $\times 21 = 8$ tons, 13 cwts.

DEFLECTION.—To find the deflection for the above load, 8 tons, 13 cwts.

$$D = m \cdot \frac{Wl^3}{EI} = \frac{5}{384} \cdot \frac{8\frac{13}{20} \times 249^3}{13500 \times 315.3}$$
$$= 0.4 \text{ inch}$$
$$= \tfrac{1}{50} \text{ inch per foot of span approximately.}$$

This result may be considered to be satisfactory.

EQUIVALENT DISTRIBUTED LOAD.—Had the load of 8 tons, 13 cwt. been applied in a concentrated form at the centre of the beam the maximum BM would have been equal to $\frac{Wl}{4}$, instead of $\frac{Wl}{8}$, as above. Thus the maximum BM produced by the concentrated load would be equal to the maximum BM produced by a distributed load of twice the weight. The latter is sometimes called the equivalent distributed load, which in this case $= 17$ tons, 6 cwts. By referring to the table it is seen that the section necessary to support this load is one 15×6 inches, weighing 59 lbs. per feet run.

JOIST WITH TWO CONCENTRATED LOADS.—Select from the table on page 72 a joist to support two concentrated loads over an effective span of 20 feet, as shown in Fig. 75.

To find the reaction at B—

$$R \times 20 = 15 \times 12 + 4 \times 5 = 200.$$
$$\therefore R = 10 \text{ tons.}$$

It is evident that maximum BM occurs under the load of 15 tons, and $= 10 \times 8 = 80$ foot-tons.

Let W equal the equivalent distributed load; that is to say, the distributed load that will produce an equal maximum bending moment, then $\frac{Wl}{8} = \frac{W \times 20}{8} = 80$.

$\therefore$ W $= 32$ tons.

$\therefore$ The required section is 18×7 inches, weighing 75 lbs. per foot run.

SHEAR.—The effect of shear must not be overlooked,

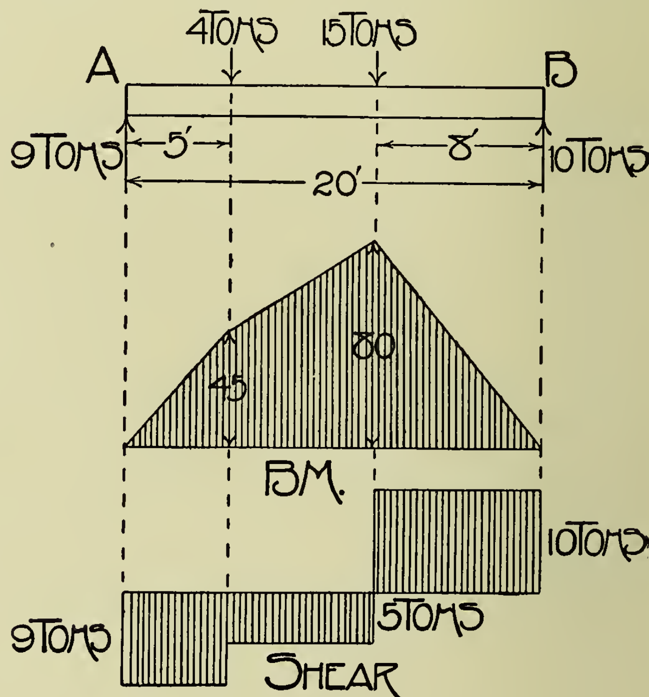


FIG. 75.

particularly in the case of a short beam with a large concentrated load, as well as in the case of a continuous beam; but for ordinary cases it will be unnecessary to consider it.

We will examine the effect of shear in the case given in the last paragraph. It is seen in Fig. 75 that at the point where the maximum BM occurs there also there is maximum shear, and that the shear at this point is 10 tons.

Referring to Fig. 51, it is seen that a large portion of the metal in the web bears only a small proportion of the stress that it is capable of bearing. It will be true enough for practical purposes, if we regard the unshaded part of this figure as metal free to resist the shear stress. This unshaded portion is seen to be approximately one half of the area of the web, which in the case under consideration $= 18 \times .55 \times \frac{1}{2} = 5$ inches.

This gives a shear stress intensity of $\frac{10}{5} = 2$ tons per square inch. This would evidently be safe; but it

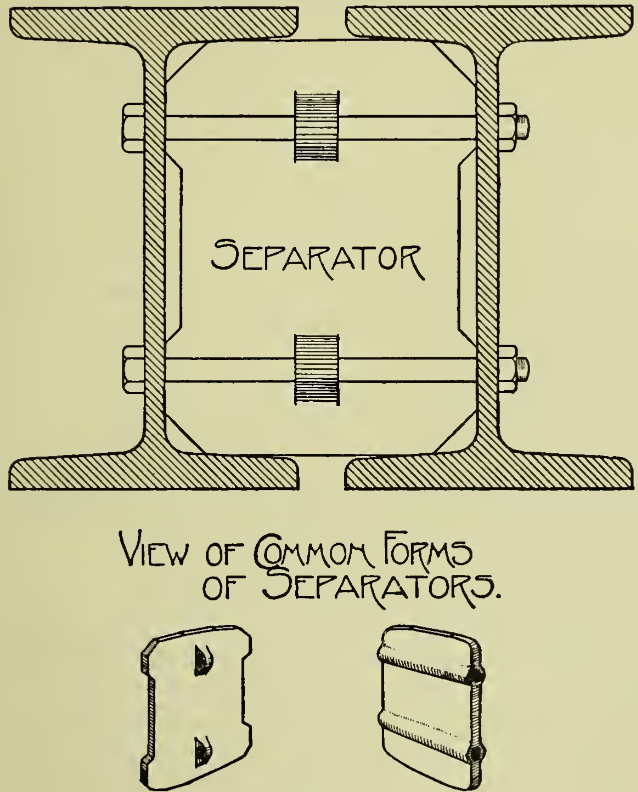


FIG. 76.

would be well, none the less, to employ four pairs of stiffeners, one pair under each load and one pair at each abutment.

COMPOUND BEAMS.—Two joists side by side bolted together at intervals (Fig. 76) are very largely used for supporting walls. Besides being stiffer than a single joist, they give a wider supporting surface and have less depth. They are connected together by separators, which should be used at intervals of 5 to 10 feet. These act as stiffeners, and should be placed where

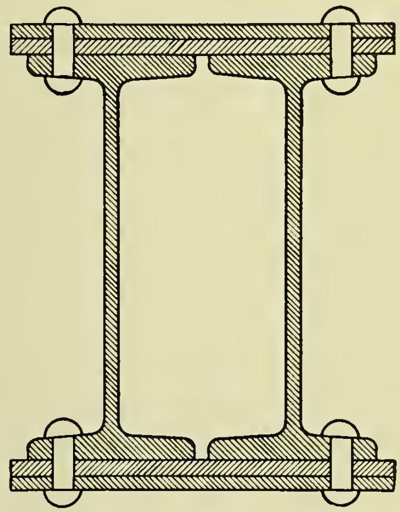


FIG. 77.

each concentrated load is applied as well as at the bearings. Gas tubing is very often used as a separator with the bolt passing through it, but this does not add stiffness to the joists.

Beams made up as shown in Fig. 77 are also largely used. Their strength may be calculated in the same way as is employed for a plate girder, or the moment of inertia may be employed.

CHAPTER VI

RIVETED JOINTS

RIVETED joints should be carefully designed so as to weaken the structure as little as possible without the use of an unnecessary amount of material or labour.

To take an example, select a steel bar to withstand a pull of 20 tons, and design a riveted joint in the same; and base the calculation on the maximum safe stresses given at the end of Chapter IV.

Without allowing for any deductions, a bar of 3 square inches section, or say $6 \times \frac{1}{2}$ inches, would contain nearly enough metal for $\frac{20}{6\frac{1}{2}} = 3$ approximately. But in forming the joint it will be necessary to weaken the bar to the extent of at least one rivet hole. Therefore

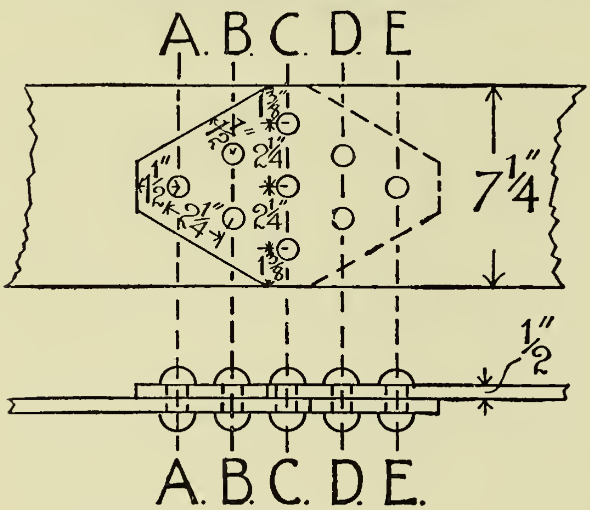


FIG. 78.

select a section $7\frac{1}{4} \times \frac{1}{2}$ inches, it being noted that rivets up to $\frac{7}{8}$-inch diameter may be riveted by hand, while, when hydraulic or pneumatic power is employed rivets of as large a diameter as $1\frac{1}{4}$ inch may be used.

The selection of the best size of rivet is largely a matter of experience. One rule of thumb says that for plates under $\frac{1}{2}$-inch thick the diameter of rivet may equal twice the thickness of plate; for plates $\frac{1}{2}$-inch thick or more rivets may be $1\frac{1}{2}$ time thickness of plate. It is therefore permissible to use $\frac{3}{4}$-inch rivets in this case. The joint is shown as a lap joint in Fig. 78. Using steel rivets, the resistance to shear of one rivet $= (\frac{3}{8})^2\pi \times 5 = 2.2$ tons; and therefore the number of rivets necessary to transmit the stress from one side of the joint to the other $= \frac{20}{2.2} = 9$ rivets.

It is now necessary to investigate the strength of the joint at its weakest parts,—that is, along the lines AA, BB, etc., where metal has been removed for the insertion of rivets. At AA the strength = the tensional strength of the bar minus the quantity removed for one rivet. Now, the nominal size of the rivets is $\frac{3}{4}$-inch diameter, but it is usual to make the holes to receive these $\frac{1}{16}$ inch larger; therefore the diameter of the hole $= \frac{13}{16}$ inch; and the metal left to resist tension $= 7\frac{1}{4} - \frac{13}{16} = 6\frac{7}{16}$; therefore the strength here $= 6\frac{7}{16} \times \frac{1}{2} \times 6\frac{1}{2} = 20.9$ tons. This is also the strength at EE. The actual pull to be resisted is only 20 tons. Consequently there is metal enough at AA and EE.

For the joint to fail at BB the lower plate must break along this line, besides which the rivet at A must shear. Therefore the strength here

$= (7\frac{1}{4} - 2 \times \frac{13}{16})\frac{1}{2} \times 6\frac{1}{2} + 2.2.$

$=$ strength of lower bar along BB + strength of rivet $= 18.3 + 2.2 = 20.5$ tons. And this is also the strength at DD.

For the joint to fail at CC, one of the plates must break along this line and three rivets must shear. Therefore strength here $= (7\frac{1}{4} - 3 \times \frac{13}{16})\frac{1}{2} \times 6\frac{1}{2} + 3 \times 2.2 = 22$ tons. Thus it is seen that the joint is strong enough for its purpose throughout.

The distance from the edge of a rivet hole to the edge of the bar must be at least equal to the actual diameter of the hole. The "pitch"—that is, the distance from centre to centre of rivet holes—should not be less than 2 inches.

If the holes be punched, the strength of the metal immediately adjoining the holes formed is appreciably affected, so that it is sometimes the custom in calculating the strength of the metal between the holes to add $\frac{1}{16}$ inch to the actual diameter of the holes or $\frac{1}{8}$ inch to the normal diameter of the rivets when deducting the metal thus removed from the gross section of the bar. If the holes be drilled the metal is not thus affected, but the process is more expensive. On the other hand, the rivets, on cooling, contract and draw the plates together, which adds considerably to the strength of the joint, but the extent of this effect is uncertain, and is therefore not relied upon.

When holes are punched the metal round the holes is slightly raised on one side and hollowed on the other, so that the plates will not fit together so closely as when the holes are drilled. This fact, together with the sharpness of the edges of the drilled holes,

probably accounts for the statement that rivets are rather more easily sheared in drilled than in punched work. Thick plates should always be drilled in good work.

SINGLE COVER PLATE.—If the bars be butt jointed and a single cover plate used to complete the joint, the cover plate may be looked upon as a link, the load being transmitted to it from one bar by a joint precisely similar to that calculated for above. The load is again transmitted from the cover plate to the other bar by another similar joint. The number of rivets in this case is twice as many as in the preceding case (Fig. 79).

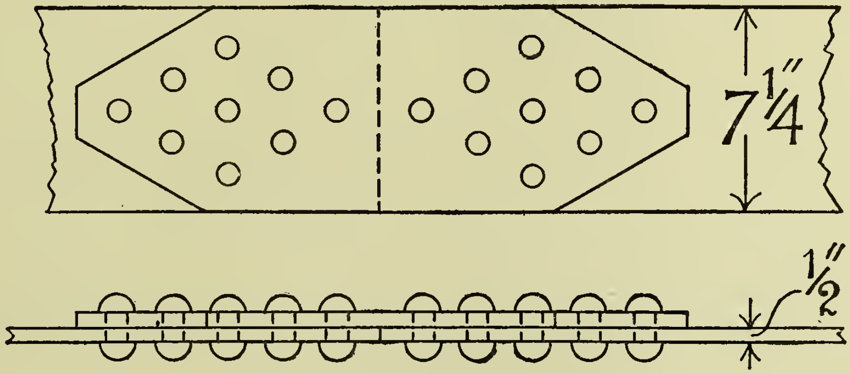


FIG. 79.

If now the cover plate be made $\frac{5}{8}$-inch thick (Fig. 80), three rivets may be inserted at AA and BB, thus making the joint shorter, for the strength of the cover plate at AA or BB $=(7\frac{1}{2}-3\times\frac{3}{4})\frac{5}{8}\times 6\frac{1}{2}=20.3$ tons.

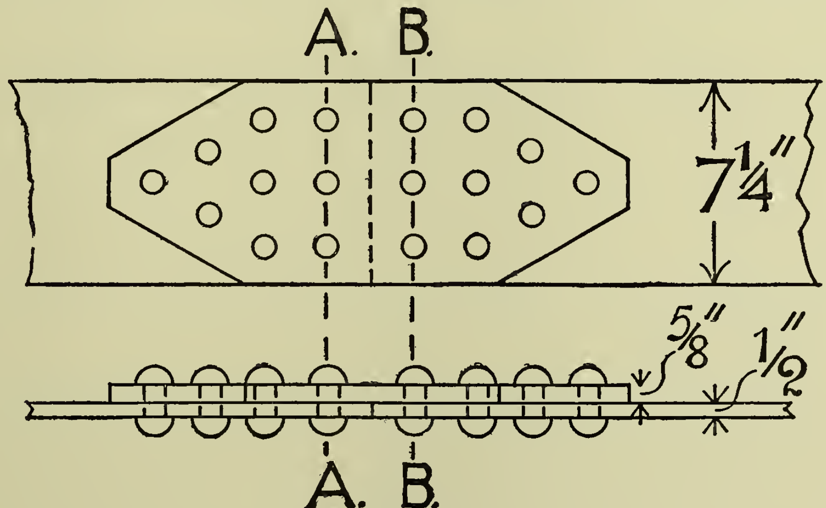


FIG. 80.

In any case the cover plate should be made slightly thicker than the parts joined.

DOUBLE COVER PLATES.—If a cover be used on either side of the joint the thickness of each would be half that of the cover in the last example. Each rivet now offers resistance to shearing between the bar and the top cover plate, and between the bar and the bottom cover plate. Thus each rivet is theoretically capable of offering twice as much resistance as before. Actually the strength of rivets in double shear is not quite twice their strength in single shear, particularly in the case of thin plates. By some, double shear is taken at $1\frac{1}{2}$ time single shear; but $1\frac{3}{4}$ time is probably a more correct value.

Thus the number of rivets necessary to join the same bars as before but with a double cover $=\frac{9}{1\frac{3}{4}}=5$. The double cover is greatly to be preferred, and when practicable should always be used in preference to the single cover plate.

BEARING.—There is still another way in which the joint taken above may fail,—that is to say, if the "bearing stress" of the rivets against the sides of the holes is too great. Fig. 81 shows the effect of too great a bearing stress in the case of a single rivet. Bearing may be described as a local compression, but the resistance to bearing is considerably more than the resistance to simple compression, and may be taken as $1\frac{1}{2}$ time the latter. The area of metal exposed to this stress = diameter of rivet × thickness of plate. We will now

investigate the bearing stress in the joint with the double cover taken in the last paragraph.

Area opposed to bearing given by each rivet $= \frac{3}{4} \times \frac{1}{2}$ inch.

$\therefore$ As the total pull = 20 tons, and there are 5 rivets, the bearing stress per square inch $= \dfrac{20}{\frac{3}{4} \times \frac{1}{2} \times 5} = 10\frac{2}{3}$ tons. This might be considered as excessive. Taking the safe bearing stress for steel as 9 tons per square inch, the necessary number of rivets $= \dfrac{20}{9 \times \frac{3}{4} \times \frac{1}{2}} = 5.9$, say 6 rivets.

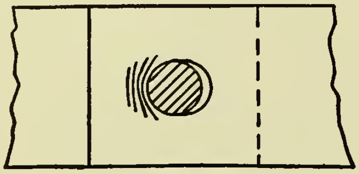


FIG. 81.

The joint thus amended is shown in Fig. 82.

The joints shown in Figs. 78, 79, and 80 are evidently safe against bearing stress.

As the rivets are arranged in Fig. 82 the bar may fail along the bent line BB if the row of rivets

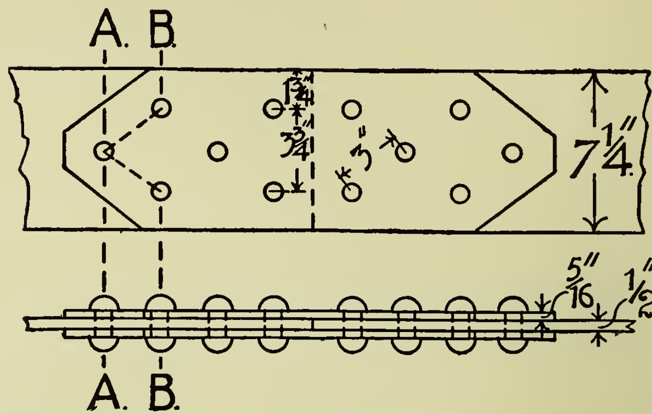


FIG. 82.

are placed too closely behind one another. There must be at least as much metal along line BB as at AA.

SAFE STRENGTH OF STEEL RIVETS IN STEEL PLATES.

Diameter of Rivet.	Single Shear at 5 Tons per Square Inch.	Double Shear 1.75 time Single Shear.	Bearing Resistance at 9 Tons per Square Inch. Thickness of Plates (in Inches).											
			$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1
Inch.														
$\frac{1}{2}$	0.98	1.72	1.41	1.69	1.97	2.25	2.53	2.81	3.09	3.37	3.66	3.94	4.22	4.50
$\frac{5}{8}$	1.53	2.68	1.76	2.11	2.46	2.81	3.16	3.52	3.87	4.22	4.57	4.92	5.27	5.62
$\frac{3}{4}$	2.21	3.87	2.11	2.53	2.95	3.37	3.80	4.22	4.64	5.06	5.48	5.91	6.33	6.75
$\frac{7}{8}$	3.01	5.26	2.46	2.95	3.45	3.94	4.43	4.92	5.41	5.91	6.40	6.89	7.38	7.87
1	3.93	6.87	2.81	3.37	3.94	4.50	5.06	5.62	6.19	6.75	7.31	7.87	8.44	9.00
$1\frac{1}{8}$	4.97	8.70	3.16	3.80	4.43	5.06	5.70	6.33	6.96	7.59	8.23	8.86	9.49	10.12

CHAPTER VII

PLATE GIRDERS

FIG. 83 shows a typical section of a plate girder. It is built to the required section with plates, while angles are used to make the connections, and the whole is riveted together.

The rivets should thoroughly fill the rivet holes, and if this be properly done in the compression flange the rivets will bear their share of compression and the flange will hardly be affected by their insertion as regards its compressional resistance. In the tension flange it is different, for here the flange is weakened to the extent of the number of rivet holes in its cross section.

The stress area for this section is shown in Fig. 84. It is seen that the web takes a very small proportion of the stress, while the flanges bear a stress which is almost equal to the limiting stress throughout their entire section. Thus it will be very near the truth if we consider the stress area to be equal to the area of the flanges + the area of the horizontal arm of the angles, as shown in Fig. 85. The effective depth may be taken as the height of the web, or the distance between the flanges, which, for girders where the flanges are of moderate thickness, will not be far from the truth, and will be on the safe side. It has been shown that the shear stress has its greatest intensity at the neutral axis of the beam, and as the web was omitted when considering the tension and compression stresses we may consider it to be free to resist shear. In other words, we design the flanges with sufficient area of cross section to resist the tensional and compressional stresses, and the web of sufficient area to resist the shear stresses, while the depth is considered equal to the distance between the flanges.

These assumptions may appear rather sweeping, but the problem is thus greatly simplified, and it must be borne in mind that the simpler the problem the less risk there is of faulty measurements and calculations ; also, on account of irregularities of quantity in the actual structure, the minutest calculations can be only approximate, while irregularities are covered by a large factor of safety.

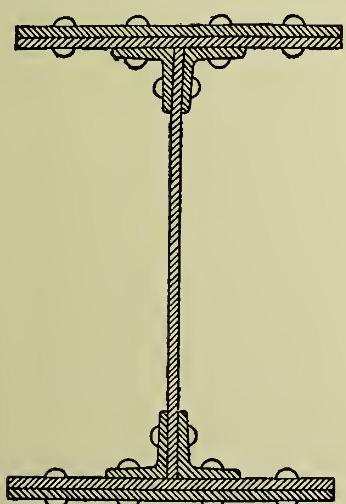


FIG. 83.

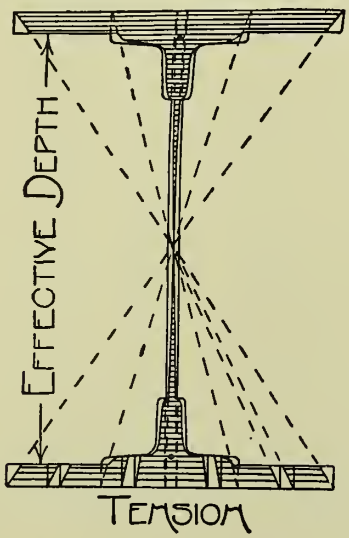


FIG. 84.

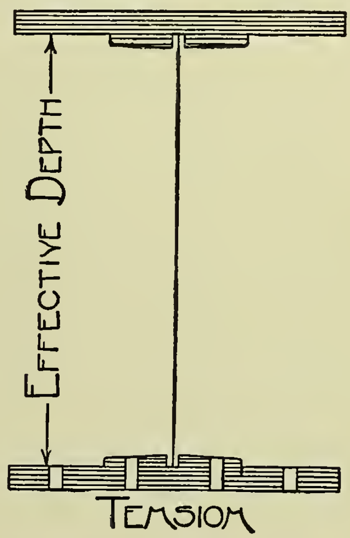


FIG. 85.

In a rolled steel joist, for sake of stiffness the web is made of considerable thickness, and if its strength were calculated in the above manner the result in most cases would be very different.

Under certain conditions it may be necessary to design a girder of this type of small depth and with thick flanges, in which case it will be well to examine the stress area in the usual way.

DESIGN.—The calculations necessary in designing a structure cannot be said to be complete until every possible effect and combination of effects that can be produced by any possible arrangement of loads has been considered. Experience will show at once what calculations are necessary and what are unnecessary, but without this experience no consideration must be

put aside until the designer has entirely satisfied himself that it will not affect his design.

By way of example, it may be assumed that it is desired to design a plate girder to carry a uniformly distributed load of 2 tons per foot run over a clear span of 30 feet 6 inches, and having a concentrated load of 20 tons at 12 feet 3 inches from one end (Plate III.).

Depth of Girder.—The depth of plate girders is usually made from $\frac{1}{8}$ to $\frac{1}{16}$ of span. $\frac{1}{12}$ of span is a very common depth, and in most cases is probably about the most economical that can be adopted. In this case the depth of web may be taken to be 32 inches.

Width of Girder.—The usual width is from $\frac{1}{20}$ to $\frac{1}{30}$ of span, and should never be less than $\frac{1}{40}$ unless the girder receive substantial lateral support. It may be assumed to be 16 inches, or half the depth and $\frac{1}{24}$ the span.

Abutments.—Allowing for a 3-inch chamfer at each abutment, the actual clear span becomes 31 feet. Then the greatest load brought upon either abutment $=31$ tons from the distributed load $+\frac{18}{30}$ of concentrated load + half-weight of girder + that part of distributed load which actually comes over the abutment. We will assume that the girder will weigh approximately 5 tons. Then if l = length in feet of bearing of girder, the total load on that bearing $=31+\frac{18}{30}\times 20+2\frac{1}{2}+l\times 2$ $=45\frac{1}{2}+2l$ tons. If it be assumed that the stone forming the girder pad is capable of sustaining a safe load of 15 tons per square foot, and the width of girder = 16 inches $=1\frac{1}{3}$ foot, the necessary length of the bearing $l=\dfrac{45\frac{1}{2}+2\,l}{15+1\frac{1}{3}}$.

$\therefore\ l=2$ feet 6 inches approximately.

It may be assumed that the abutment is to be built of good bricks in cement mortar, on which a safe load of 6 tons per square foot may be allowed. The stone then must have an area of $\frac{15}{6}=2\frac{1}{2}$ times the bearing of girder, which $=2\frac{1}{2}\times 1\frac{1}{3}\times 2\frac{1}{2}=8\frac{1}{3}$ square feet, or it may be 3×3 feet and of suitable depth in brick dimensions, say 1 foot 9 inches deep. To obtain an even bedding a sheet of lead will be inserted between the girder and the stone.

Stress Diagram.—It is next required to know the distribution of stress throughout the girder, to discover which, diagrams are drawn in the manner described in reference to H, Fig. 64. These diagrams are given at right-hand bottom corner of Plate III.

The *Effective Span* = clear span + chamfers + depth of one bearing = 33 feet 6 inches. The maximum flange stress in this case is seen to be at the point of application of the concentrated load, and the diagram may be checked by calculating the stress at this point.

Reaction of left-hand abutment $=33\frac{1}{2}$ tons from distributed load $+\,2\frac{1}{2}$ tons from weight of girder $+\,11.8$ tons from concentrated load $=47.8$ tons (see shear diagram).

$\therefore$ Maximum BM $=47.8$ tons $\times$ 13 feet 9 inches $-\,27.5$ tons $\times\,\dfrac{13\text{ ft. }9\text{ ins.}}{2}$ (due to distributed load) $-\,\dfrac{13\frac{3}{4}}{33\frac{1}{2}}\times 5$ tons $\times\,\dfrac{13\text{ ft. }9\text{ ins.}}{2}$ (due to weight of girder) $=454$ foot-tons.

$\therefore$ Dividing by depth of girder, maximum stress $=\dfrac{454\times 12}{32}=170$ tons.

Flanges.—Plates $\frac{1}{2}$ to $\frac{3}{4}$-inch thick are generally preferred, while plates up to $\frac{7}{8}$ or 1-inch thick may be used if necessary. Tables showing sizes of plates and sections are attached at the end of this Chapter. The use of plates of thicknesses running in $\frac{1}{8}$'s inch, as $\frac{1}{2}$, $\frac{5}{8}$, $\frac{3}{4}$ inch, may perhaps secure a more ready delivery when the amount is small and time is short; while often considerable time may be saved by the use of common sizes of angles, such as 4×4 inches, $3\frac{1}{2}\times 3\frac{1}{2}$ inches, 3×3 inches, or 4×3 inches. In selecting the thickness of plates to be used, uniformity should be aimed at as far as economy will allow, this being merely a matter of convenience in the manufacturers' shops.

The selection of the angles to be used depends largely upon convenience in riveting. The width of the side of the angles minus the thickness should be at least equal to three times the diameter of the rivets to be used; there should also be a width of at least three times the diameter of rivets between edge of angle and edge of plate; but, on the other hand, this width should not be excessive, say not more than eight times the thickness of the plate which it immediately adjoins. The thickness of the angle should be about the same as the average thickness of the plates used in the flange; $4\times 4\times\frac{1}{2}$-inch angles may here be used.

In calculating the strength of a rolled steel joist a maximum fibre stress of $7\frac{1}{2}$ tons per square inch was allowed, but in that case the section was homogeneous, while in the case at present under consideration the section is built up of many plates more or less rigidly connected, and the strength of the structure depends to a considerable extent upon the quality of the workmanship expended upon it. Under these circumstances the calculations should be based upon the maximum safe loads allowed by the Board of Trade,—namely, $6\frac{1}{2}$ tons per square inch both in tension and compression.

The necessary quantity of metal in each flange to resist the stress of 170 tons, as found in the last paragraph, $=\dfrac{170}{6\frac{1}{2}}=26.16$ square inches.

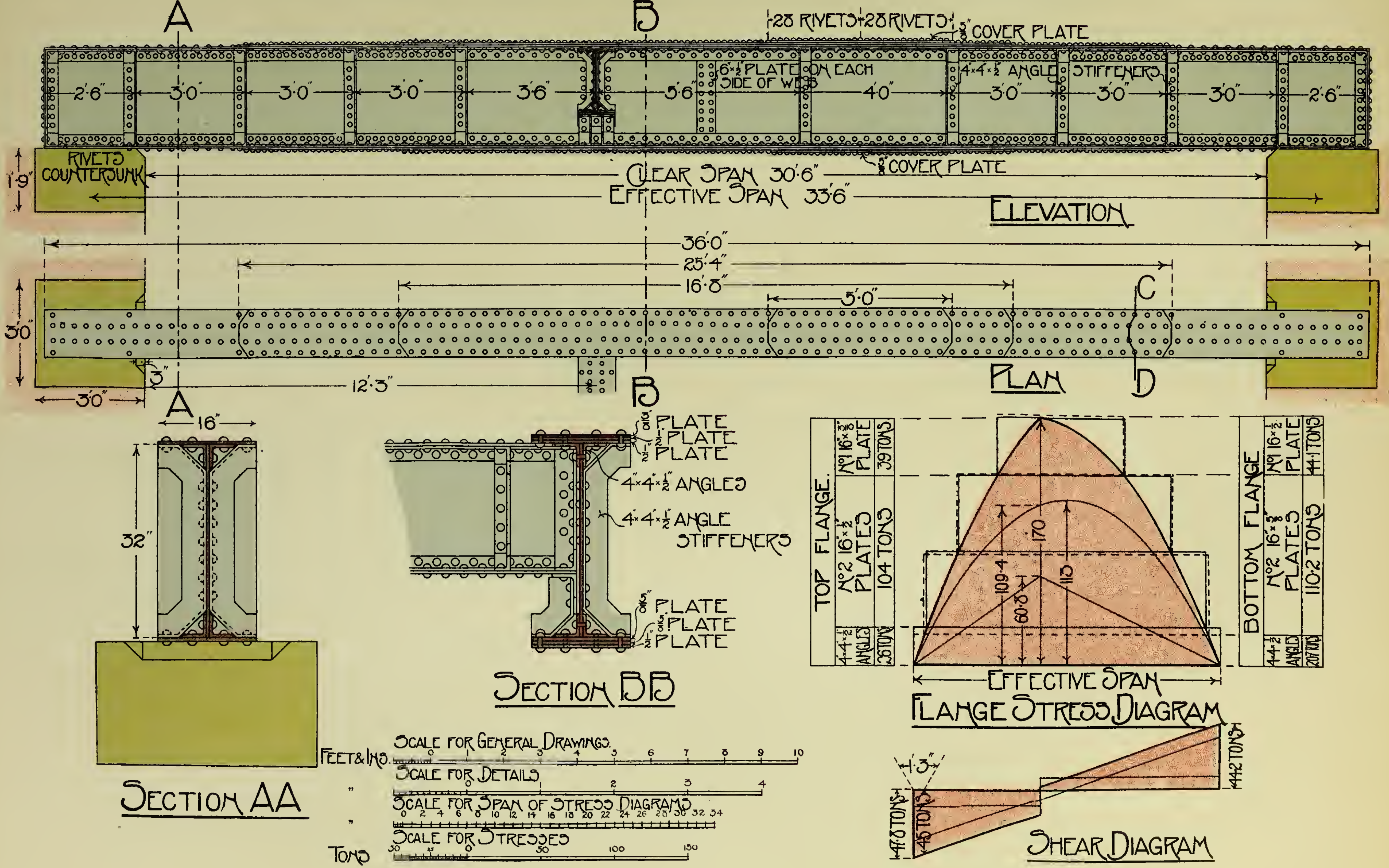
The horizontal arm of the angles, as before stated, may be considered to take its share of the stress. In the upper or compression flange the area of section of this part of the angles $=2\times 4\times\frac{1}{2}$ inches $=4$ square inches.

Therefore in this upper flange the necessary sectional area of the plates $=26.16-4=22.16$ square inches; and as it has been decided that the plates are to be 16 inches wide, the necessary thickness $=\dfrac{22.16}{16}=1.4$ inch $=1\frac{3}{8}$ inch approximately.

In the lower or tension flange the metal removed for

DESIGN FOR A STEEL PLATE GIRDER

PLATE III



riveting must be allowed for. It may be assumed that $\frac{3}{4}$-inch rivets are to be used, and that the holes are to be drilled and of $\frac{13}{16}$-inch diameter. The sectional area of the horizontal arm of the angles will then be $2 \times (4 - \frac{13}{16} \text{ inch}) \times \frac{1}{2} \text{ inch} = 3.19$, one rivet being necessary to connect each angle to the flange.

Therefore in the lower flange the necessary sectional area of the plates $= 26.16 - 3.19 = 23$ square inches.

As was shown in Chapter VI., the necessary number of rivets may be placed zigzag, weakening the parts riveted to a less extent than if placed in straight lines across the member; and there is no reason why they should not be so placed in the flanges of a plate girder, as shown on the plan in Plate III., although they are very generally placed in straight lines across them. However, if they are placed zigzag, and allowance is made for the metal thus saved, it must be observed that this form of riveting is not altered at any point in the length of the girder unless there is evidently a superfluous quantity of metal at this part. As was also pointed out in Chapter VI., the section of the parts joined may be at its minimum in a zigzag line, as at CD in plan of girder, and it will be found that this is so in the present case. The sectional area along this line may, in this instance, be taken as the simple cross section of the plates, less the amount of metal removed for three rivets.

Thus the effective width of the plates at this part $= 16 \text{ inches} - 3 \times \frac{13}{16} \text{ inches} = 13\frac{9}{16}$ inches.

$\therefore$ The necessary thickness $= \dfrac{23}{13\frac{9}{16}} = 1.7 = 1\frac{11}{16}$ inch approximately.

Wrought iron is now hardly ever used in girder construction. In designing a girder of this material, with maximum tension at 5 tons per square inch and compression at 4 tons per square inch, it would be found that the necessary thickness of flanges would be approximately the same for both flanges, and in practice they are always made equal.

In the smaller steel girders, for sake of simplicity and convenience of construction, the flanges are generally made of equal thickness, the necessary thickness for the tension flange being calculated, and the compression flange being made equal to it. If it were intended to make the flanges of equal thickness in the present case they would both have to be $1\frac{11}{16}$ inch thick.

As before stated, the necessary thickness of compression flange $= 1\frac{3}{8}$ inch, which might be made up of one plate $\frac{3}{4}$ inch thick and one plate $\frac{5}{8}$ inch thick, in which case the thicker plate would be placed next to the angles and the thinner plate on the outside.

Instead of this, the compression flange is shown in the Plate to be composed of two $\frac{1}{2}$-inch plates and one $\frac{3}{8}$-inch plate, while the necessary thickness of $1\frac{11}{16}$ inch in the tension flange is made up with two plates $\frac{5}{8}$ inch thick and one $\frac{1}{2}$ inch thick. It may be noticed that the plates in the tension flange have each been made $\frac{1}{8}$ inch thicker than those in the compression flange. The result of this, as will be seen, is to make the necessary lengths of the plates respectively equal in the two flanges.

The load which each plate is capable of resisting is now plotted upon the stress diagram to the same scale of tons to which the rest of the diagram is drawn. In this way a number of horizontal lines are obtained, the distance between each pair of lines representing the safe stress that may be put upon the corresponding plate. The points at which these lines cut the stress curve indicate points in the flange between which the plate next above it must extend. The full lines indicate the strength and length of the plates in the upper flange, while the dotted lines represent those of the lower flange. It is seen that the necessary length of plates in the upper flange is less than that of plates in the lower flange, but as this difference is, in this case, quite small the corresponding plates of the two flanges may be made of equal length for sake of simplicity. It is permissible to do this, for the reason that each plate has been thickened in the lower flange in proportion to the amount of metal removed in the rivet holes, thus obtaining practically the same sectional area in the respective plates of the two flanges.

According to the diagram, the plate which is next to the angles need not extend farther towards the ends of the girder than 1 foot from the centre of each bearing surface, but for the sake of rigidity and to preserve the full width of the bearing surfaces this inner plate is always carried the full length of the girder.

The second plate, according to the diagram, need not extend more than to a point 4 feet 9 inches from the centre of the left-hand bearing; but in order that the plate at this point may be taking some share of the stress it should extend at least one row of rivets beyond the position indicated on the diagram. In the girder as illustrated (Plate III.) each plate is extended sufficiently to receive at least six rivets beyond the points shown by the diagram.

Cover Plates.—On reference to page 85 it will be seen that all the plates may be obtained in one length, but for sake of example it may here be assumed that the longest plate has to be made out of two lengths. If the joint be made at the centre of the girder the two plates will be of minimum length, but the plate will be cut at a point where it is at its maximum stress. Although the joint should be designed to be, if anything, stronger than the parts joined, yet it will be well to form it where the intensity of stress is not at its maximum. Nevertheless the centre is a very usual position for the joint. In the present case it is shown at a point 4 feet 2 inches from the centre.

Assuming the rivets to be $\frac{3}{4}$-inch steel rivets with a safe shear stress of 5 tons per square inch, the plates joined have a strength of 55 tons. The strength of one rivet in single shear $= 2.2$ tons. Therefore the number of rivets necessary in cover plate on either

side of joint $= \frac{55}{2.2} = 25$ rivets; but for convenience sake 28 rivets or 7 rows will be used. The thickness of the cover plate must be at least as great as that of the plate joined.

The joint might be made just at the termination of the outside plate, which would then be extended by another 26 rivets, thus avoiding the use of a

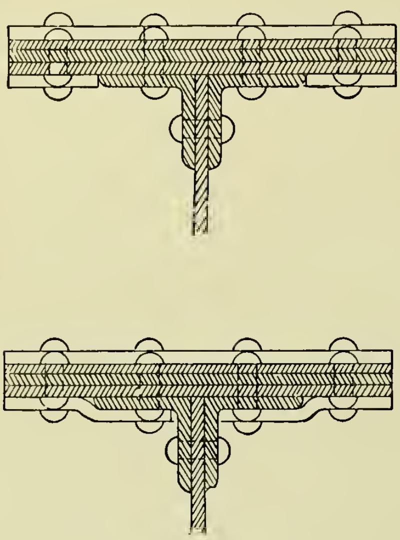


Fig. 86.

cover altogether; but this is open to the same objection as is given for the case when the joint is at the centre.

As stated in the last chapter, the use of double covers is always advisable, and Fig. 86 shows how these may be arranged. Their use is particularly advisable if covering joins in more than one plate.

If it be required to make a join in the angles it will be done as shown in Fig. 87, using a cover of sectional

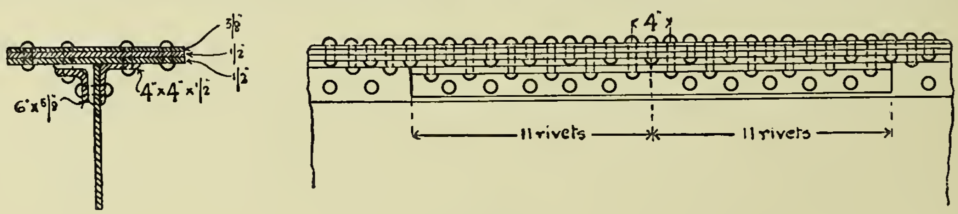


Fig. 87.

area equal to that of the angle and forged to fit on the under side of it. The sectional area of the angle $= 7\frac{1}{2} \times \frac{1}{2} = 3\frac{3}{4}$ inches, and to form the cover a piece of steel $6 \times \frac{5}{8}$ inches might be used. The number of rivets on either side of the joint $= \frac{3\frac{3}{4} \times 6\frac{1}{2}}{2.2} = 11$ rivets.

Web.—As before stated, it is assumed that the sole function of the web is to resist shear. According to the diagram on Plate III., the maximum shear is at the centre of the left-hand bearing, where it is shown to be 47.8 tons; however, as the reaction is imparted to the girder throughout the length of the bearing, the maximum shear actually occurs at the edge of the bearing, where it is 45 tons.

The stresses in the web are highly complex. It cannot be accurately designed by simple calculation, and its proportion must depend largely upon judgment. Besides the shear stress, it has to resist a direct compressive stress due to the distributed load. It is usual to make the web of sufficient thickness to give a shear stress of 3 tons per square inch, rigidity being added to it and to the girder in general by the use of "stiffeners" placed at intervals according to judgment. Thus the necessary thickness of web $= \frac{45}{32 \times 3} = \frac{1}{2}$ inch. The number and position of stiffeners will be discussed later.

It will be convenient to form the web in two pieces of equal length with a joint at the middle. The shear at this point is seen to be 8 tons. The resistance to *bearing* of one rivet through the web $= \frac{1}{2} \times \frac{3}{4} \times 9 = 3.4$ tons. The bearing of the rivets is taken as governing their number instead of their shearing value, as the rivets will be in double shear and will in this case give a greater value. Thus theoretically $\frac{8}{3.4} = 3$ rivets are required on either side of the joint. For the sake of stiffness the joint will be formed with two cover plates of the same thickness as web,—that is, $\frac{1}{2}$ inch thick and 6 inches wide, and the same pitch for the rivets will be used as in the rest of the girder.

The shear diagram shows that a thinner web plate might be employed along the central part of the girder. It is, however, seldom desirable to use more than one thickness in a girder with a solid web such as this.

Pitch of Rivets.—It has been already stated that horizontal shear per foot run = vertical shear per foot run. A simple example will demonstrate the truth of this. Fig. 88 represents a girder of construction similar to the one under consideration, and loaded with a single concentrated load. The stress in the flanges at any point X $= \frac{R_1 m}{d}$. Considering the upper flange on the left-hand side of the section at X, this force $\frac{R_1 m}{d}$ acts in the direction of R_1 to W. Assume for an instant that the stress in flange between R_1 and X is constant and equal to $\frac{R_1 m}{d}$. Now, to resist this force there must be a shearing stress throughout

the length m, between the flange and the web, equal to $\frac{R_1 m}{d}$ which $= \frac{R_1 m}{d} \div m = \frac{R_1}{d}$ per foot run, and this is the horizontal shearing stress per foot run at point X.

The total vertical shearing stress at this point $= R_1$, which is also equal to $\frac{R_1}{d}$ per vertical foot run of web.

Thus at all points throughout the length of the girder there is a horizontal shearing force between the flange and the web equal to the vertical shearing force per foot run at the same point.

This horizontal shear has to be resisted by the rivets in the vertical arm of the angles.

As a matter of fact, this horizontal shearing stress

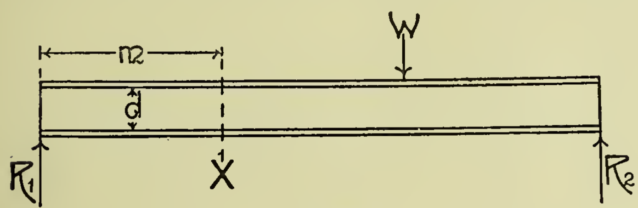


Fig. 88.

will be slightly less than is stated above, as the web itself takes a small portion of the flange stress. On the other hand, the shear on the rivets of the upper flange is slightly increased by the direct weight of a distributed load as shown in Fig. 89, where AB = the horizontal shear per foot run, BC = vertical shear due to distributed load per foot run, and AC is the resultant shear.

It will be near enough for the present purpose to assume the truth of the original statement.

Besides the shearing force between the angles and the web there is a shearing force between the plates and the angles; the latter stress is less than the former in the ratio of the sectional area of the plate

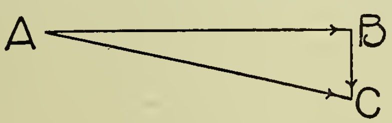


Fig. 89.

or plates, to the sectional area of the whole flange including the angles.

In the present case, as before stated, the maximum shear = 45 tons, which $= \frac{45 \times 12}{32} = 17$ tons per foot run, —and this is the shear between the angles and the web.

The shear between the plate and the angles (there is only one plate at the point which is being considered) $= \frac{16}{8+16} \times 17 = 11\frac{1}{3}$ tons per foot run, and slightly more than this in the lower flange. To resist this shear the necessary number of rivets per foot run of flange $= \frac{11\frac{1}{3}}{2.2} = 5$; and as there are two rows of rivets which connect the plate to the angles, the greatest "pitch" or distance apart of rivets in inches $= \frac{12}{2\frac{1}{2}} = 4\frac{4}{5}$ inches.

A 4-inch pitch is the greatest that should be used in the compression flange in order to avoid the buckling of the plates, as it is generally by such buckling in the compression flange that the first signs of failure appear. The zigzag arrangement of rivets, however, considerably lessens this possibility. In the tension flange a $4\frac{4}{5}$-inch pitch might be used at the ends of the girder, increasing it to 6 inches at the centre; but for sake of uniformity and to avoid difficulty in the placing of the stiffeners the pitch is here made 4 inches throughout; and this is a very usual pitch where there is no objection to its use. 6-inch pitch is the greatest that should be used in any case, as if carried beyond this limit moisture may find its way between the plates, resulting in oxidation and the forcing apart of the plates.

It is now necessary to investigate the pitch of rivets to connect angles to web. The shear, as before stated, is here equal to 17 tons per foot run, which in double shear requires $\frac{17}{2.2 \times 1.75} = 3.8$ rivets; or, calculating by the bearing of the rivets, the necessary number $= \frac{17}{\frac{3}{4} \times \frac{1}{2} \times 9} = 5$ rivets per foot run. The latter number must therefore be taken, and the pitch $= \frac{12}{5} = 2.4$ inches, or say $2\frac{1}{2}$-inch pitch.

When the shear stress becomes 10 tons per foot run the number of rivets necessary $= \frac{10}{\frac{3}{4} \times \frac{1}{2} \times 9} = 3 = 4$-inch pitch. The shear stress becomes 10 tons per foot run, when the total shear in the web $= \frac{10 \times 32}{12} = 26\frac{1}{2}$, and by measuring on the shear diagram in Plate III. we find that the shear is equal to this at a point 9 feet 11 inches from the centre of the left-hand bearing.

The pitch in the vertical arm of the angle is therefore made $2\frac{1}{2}$ inches for a length of 11 feet at the ends, the remainder having 4-inch pitch.

By making the pitch exactly the same at both ends, work in the template shop is lessened, while the possibility of mistake is minimised.

The rivets over the bearing of the girder must be countersunk to give an even surface.

Stiffeners.—As before stated, these are employed to add stiffness to the web and girder in general. They are composed of tees or angles, as shown in the details given in Plate III., or as in Fig. 90; or of angles on either side of a piece of plate steel, termed a "gusset," as shown in Fig. 91. Of whatever form stiffeners are constructed, they must be cut and forged to fit accurately and tightly between the two flanges.

For the girder under consideration, where the width of the girder is large and four rows of rivets are employed, the type of stiffener there shown has been adopted. The forging of this form is expensive. When the girder is narrow the form shown in Fig. 90 is used, where the tee or angle merely butts against the

angles of the flange instead of being riveted to the plates. The stiffener may be forged or "joggled" so as to rise over the vertical arm of the angles, as shown on the right-hand side of Fig. 90; or a "packing-

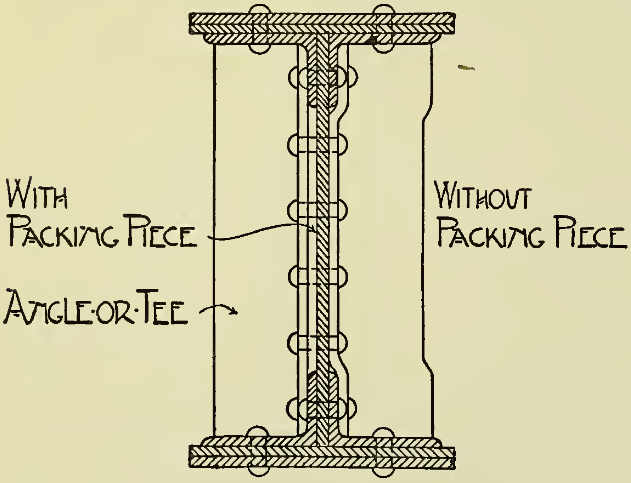


Fig. 90.

piece" may be employed between the stiffener and the web, as shown on the left. In either case the ends of the stiffeners must be made to fit tightly against the horizontal arms of the angles. The latter method is probably the most economical when the number of stiffeners is not large.

The use of a packing piece is again shown on the left of Fig. 91. This form of stiffener is stronger than the others, and is suitable for girders of slightly greater width than in the last case. Gusset pieces may also be used with angles forged to the shape shown in Plate III. In the upper part of Fig. 91 the rivets in the two arms of the angles are shown opposite one another; in the lower flange they are placed alternately. In the latter case the rivet immediately below the stiffener must be countersunk.

Position of Stiffeners.—Stiffeners must be placed where the shear stress is greatest, and at any point where a concentrated load is brought upon the girder. Thus they are always placed at the inner side of the bearing, as shown in the elevation in Plate IV.; and again, at the point where the load of 20 tons is brought upon the girder by a smaller lateral girder, as shown in the same figure; and it may be noted that there must be a sufficient number of rivets connecting these two girders to resist a shear stress of 20 tons. Thus the positions of three pairs of stiffeners are fixed. The spaces between these must be divided up by pairs of stiffeners according to judgment. They may all be placed at equal intervals, or the spacing may be increased towards the centre where the shear stress is least. The ends of the girder are closed up with plates of the same width as the girder, and attached to it by means of angles.

Deflection.—Referring to page 67, it will be seen that $\text{Deflection} = \frac{1}{8}\frac{fl^2}{Ey} = \frac{1}{8} \cdot \frac{6\frac{1}{2} \times (33\frac{1}{2} \times 12)^2}{13{,}500 \times 17} = 0.57$ inch, which evidently need not be considered.

Weight.—The weight of the girder is found by calculating the weight of plates, angles, and tees separately, adding 5 per cent. for rivets. The actual weight of this girder is thus found to be approximately $4\frac{3}{4}$ tons. The weight has been already assumed at 5 tons, which is thus very near the truth.

Box Girders.—Fig. 92 shows the section of a box girder. Calculations for this form are precisely the same as those necessary for the design of a single-web plate girder. The sole difference between a single-web girder and a box girder is that in the latter the necessary thickness of web is made up in two thicknesses of metal. Metal $\frac{1}{4}$ inch in thickness is the thinnest that may be used, and it will generally be better, in designing box girders, to make the webs not less than $\frac{3}{8}$-inch thick to allow for the increased liability to corrosion. This form of girder is stiffer than the single-web type, weight for weight; but its construction is more difficult, and, as already stated, on account of the difficulty of painting, the chances of corrosion are increased, so that the single-web type is generally preferable.

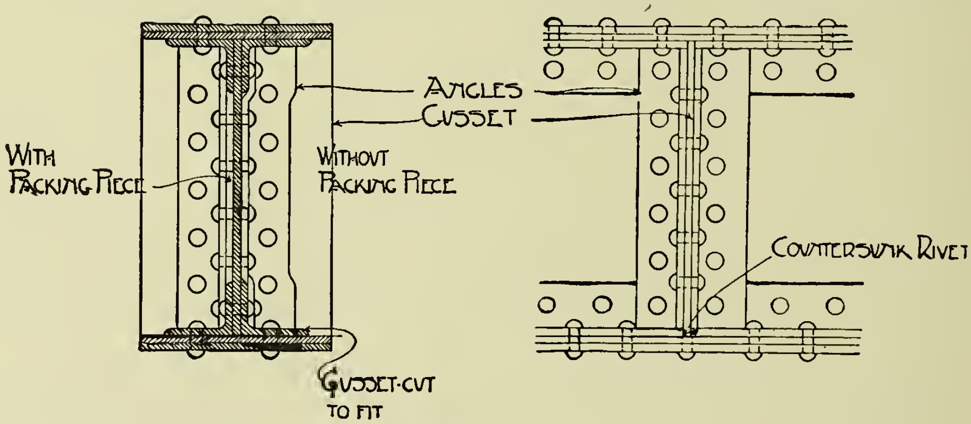


Fig. 91.

Dimensions of Plates and Sections.—Below are given extracts from the lists of various manufacturers. They are given here merely as a guide to the ordinary sizes rolled; for these particulars vary with every

manufacturer, while, by special arrangement, dimensions other than those given in manufacturers' lists may be worked to if necessary.

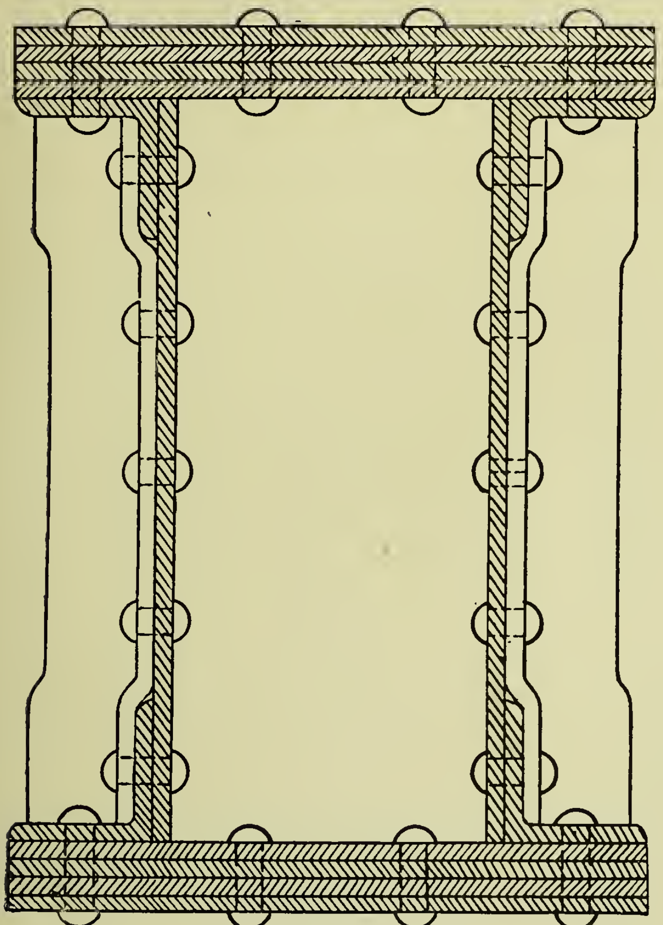


FIG. 92.

MAXIMUM DIMENSIONS TO WHICH STEEL PLATES ARE ROLLED BY MESSRS. STEWARTS & LLOYDS LTD.

Thickness.	Length.	Width.	Area.
Inches.	Feet.	Inches.	Square Feet.
$\frac{1}{8}$	25	51	68
$\frac{5}{32}$	30	56	93
$\frac{3}{16}$	40	60	120
$\frac{7}{32}$	40	60	130
$\frac{1}{4}$	45	72	180
$\frac{5}{16}$	45	72	180
$\frac{3}{8}$	40	84	120
$\frac{7}{16}$	45	94	135
$\frac{1}{2}$	50	106	200
$\frac{9}{16}$	50	110	210
$\frac{5}{8}$	60	116	220
$\frac{11}{16}$	60	116	220
$\frac{3}{4}$	60	122	230
$\frac{7}{8}$	60	122	230
1	60	122	240
$1\frac{1}{8}$	60	122	240
$1\frac{1}{4}$	60	122	240
$1\frac{3}{8}$	60	122	240
$1\frac{1}{2}$	60	122	240
$1\frac{5}{8}$	50	122	200
$1\frac{3}{4}$	45	122	190
$1\frac{7}{8}$	40	120	180
2	40	120	170

Plates cannot be rolled to both the maximum length and maximum width, but to the maximum area. The area divided by the length will give the maximum width for any plate, and *vice versâ.*

Plates of somewhat larger dimensions than those stated can be obtained by special arrangement.

As a general rule extras are charged in the following cases, but this question depends very largely upon the nature of the specification: Plates weighing over 80 cwts.; plates more than $1\frac{1}{2}$ inch or less than $\frac{3}{8}$ inch thick; plates $\frac{3}{16}$ to $\frac{9}{16}$ inch thick, having widths exceeding 54 to 90 inches; plates $\frac{9}{16}$ inch thick and upwards, having widths exceeding 96 inches.

DIMENSIONS OF FLATS ROLLED BY THE FRODINGHAM IRON AND STEEL COMPANY LTD.

Width.	Normal Thicknesses. Edges practically Square.	Greater Thicknesses. Edges slightly Rounded.
Inches.	Inch. Inch.	Inch. Inches.
$1\frac{1}{4}$	$\frac{3}{16}$ to $\frac{5}{8}$	$\frac{11}{16}$ to 1
$1\frac{1}{2}$	$\frac{3}{16}$,, $\frac{5}{8}$	$\frac{11}{16}$,, 1
$1\frac{3}{4}$	$\frac{3}{16}$,, $\frac{5}{8}$	$\frac{11}{16}$,, 1
2	$\frac{1}{4}$,, $\frac{3}{4}$	$\frac{13}{16}$,, $1\frac{1}{4}$
$2\frac{1}{4}$	$\frac{1}{4}$,, $\frac{3}{4}$	$\frac{13}{16}$,, $1\frac{1}{4}$
$2\frac{1}{2}$	$\frac{1}{4}$,, $\frac{3}{4}$	$\frac{13}{16}$,, $1\frac{1}{4}$
$2\frac{3}{4}$	$\frac{1}{4}$,, $\frac{3}{4}$	$\frac{13}{16}$,, $1\frac{1}{4}$
3	$\frac{1}{4}$,, $\frac{3}{4}$	$\frac{13}{16}$,, $1\frac{1}{4}$
$3\frac{1}{4}$	$\frac{1}{4}$,, $\frac{3}{4}$	$\frac{13}{16}$,, $1\frac{1}{4}$
$3\frac{1}{2}$	$\frac{1}{4}$,, $\frac{3}{4}$	$\frac{13}{16}$,, $1\frac{1}{4}$
$3\frac{3}{4}$	$\frac{1}{4}$,, $\frac{3}{4}$	$\frac{13}{16}$,, $1\frac{1}{4}$
4	$\frac{1}{4}$,, 1	$1\frac{1}{16}$,, $2\frac{1}{2}$
$4\frac{1}{4}$	$\frac{1}{4}$,, $\frac{3}{4}$	$\frac{13}{16}$,, $1\frac{1}{4}$
$4\frac{1}{2}$	$\frac{1}{4}$,, $\frac{3}{4}$	$\frac{13}{16}$,, $1\frac{1}{4}$
5	$\frac{1}{4}$,, 1	$1\frac{1}{16}$,, $2\frac{1}{2}$
$5\frac{1}{2}$	$\frac{1}{4}$,, 1	$1\frac{1}{16}$,, $2\frac{1}{2}$
6	$\frac{3}{8}$,, 1	$1\frac{1}{16}$,, $2\frac{1}{2}$
7	$\frac{3}{8}$,, 1	$1\frac{1}{16}$,, $2\frac{1}{2}$
8	$\frac{3}{8}$,, 1	$1\frac{1}{16}$,, $2\frac{1}{2}$
9	$\frac{3}{8}$,, 1	$1\frac{1}{16}$,, $2\frac{1}{2}$
10	$\frac{3}{8}$,, 1	$1\frac{1}{16}$,, $2\frac{1}{2}$
11	$\frac{3}{8}$,, 1	$1\frac{1}{16}$,, $2\frac{1}{2}$
12	$\frac{3}{8}$,, 1	$1\frac{1}{16}$,, 2
14	$\frac{3}{8}$,, 1	$1\frac{1}{16}$,, 2

Extras.—Sizes, under 5 inches wide, over 12 inches wide, and under $\frac{5}{16}$ inch thick; lengths, over 40 feet, or under 5 feet.

ROUNDS AND SQUARES are made by most makers approximately to the sizes given below:—

Diameter of Rounds.			Sides of Squares.		
Inches.		Inch.	Inches.		Inches.
$\frac{3}{8}$ to $1\frac{1}{8}$,	advancing by $\frac{1}{64}$ or $\frac{1}{32}$		$\frac{1}{2}$ to $1\frac{1}{2}$,	advancing by $\frac{1}{16}$	
$1\frac{1}{8}$,, $1\frac{1}{2}$	,,	$\frac{1}{16}$	$1\frac{1}{2}$,, 3	,,	$\frac{1}{8}$
$1\frac{1}{2}$,, 5	,,	$\frac{1}{8}$	3 ,, 6	,,	$\frac{1}{4}$
5 ,, 8	,,	$\frac{1}{4}$			

Extras are generally charged both on rounds and squares for sizes under $\frac{5}{8}$ to $\frac{1}{2}$ inch or over 3 to $3\frac{3}{4}$ inches, and also for lengths less than 5 or more than 25 feet.

DIMENSIONS OF ANGLES AND TEES ROLLED BY THE FRODINGHAM IRON AND STEEL COMPANY LTD.

Angles. Equal Sides.			Angles. Unequal Sides.						Tees.			
Size.	Min. Thickness Rolled.	Max. Thickness Rolled.	Size.	Min. Thickness Rolled.	Max. Thickness Rolled.	Size.	Min. Thickness Rolled.	Max. Thickness Rolled.	Size. Flange. Web.	Thicknesses Rolled.		
Inches.	Inch.	Inch.	Inches.	Inch.	Inch.	Inches.	Inch.	Inch.	Inches.	Inch.	Inch.	Inch.
1¼ × 1¼	1/8	5/16	**1¼ × 1**	1/8	1/4	**5 × 3**	1/4	5/8	2 × 1½	1/4	...	3/8
1½ × 1½	1/8	3/8	**1¾ × 1½**	1/8	3/8	**5 × 3½**	5/16	5/8	**2 × 2**	1/4	...	3/8
1¾ × 1¾	1/8	3/8	**2 × 1½**	1/8	3/8	**5 × 4**	5/16	11/16	2½ × 2	3/8	...	1/2
2 × 2	1/8	3/8	2½ × 1½	5/32	7/16	**5 × 3**	6/20	1/2	**2½ × 2½**	1/4	3/8	1/2
2¼ × 2¼	5/32	1/2	**2½ × 2**	5/32	7/16	**5½ × 3½**	6/20	1/2	**3 × 2½**	...	3/8	1/2
2½ × 2½	3/16	9/16	3 × 1½	5/16	3/4	6 × 3	5/16	3/4	**3 × 3**	...	3/8	1/2
2¾ × 2¾	3/16	1/2	**3 × 2**	3/16	1/2	**6 × 3½**	5/16	3/4	3½ × 3	3/8	...	1/2
3 × 3	3/16	5/8	**3 × 2½**	3/16	1/2	**6 × 4**	5/16	3/4	**3½ × 3½**	3/8	...	1/2
3¼ × 3¼	5/16	5/8	**3½ × 2½**	1/4	5/8	**6½ × 3½**	5/16	3/4	4 × 2½	3/8	1/2	...
3½ × 3½	1/4	3/4	**3½ × 3**	1/4	5/8	**6½ × 4½**	11/32	3/4	**4 × 3**	3/8	1/2	...
4 × 4	5/16	3/4	**4 × 2½**	1/4	5/8	7 × 3	11/32	3/4	**4 × 4**	3/8	1/2	5/8
4½ × 4½	5/16	3/4	**4 × 3**	1/4	5/8	**7 × 3½**	11/32	3/4	**4 × 5**	...	1/2	5/8
5 × 5	3/8	7/8	**4 × 3½**	1/4	5/8	9 × 3½	7/16	3/4	5 × 2½	3/8	1/2	...
5½ × 5½	3/8	7/8	**4½ × 3**	1/4	5/8				**5 × 3**	3/8	1/2	5/8
6 × 6	7/16	7/8	**4½ × 3½**	1/4	5/8				**5 × 3½**	3/8	1/2	5/8
7 × 7	1/2	7/8	4½ × 4	5/16	5/8				**5 × 4**	1/2	5/8	...
									5 × 5	1/2	5/8	...
									6 × 3	3/8	1/2	5/8
									6 × 3½	1/2	5/8	...
									6 × 4	1/2	5/8	...

Extras—Under 6 or over 12 united inches, under 5/16 inches thick or under 5 feet in length.
Note.—Where the size of the tee is printed in heavy type the section is British standard.

DIMENSIONS OF CHANNELS ROLLED BY THE FRODINGHAM IRON AND STEEL COMPANY LTD.

Size.			Weight per Foot.	Thickness.		Size.			Weight per Foot.	Thickness.	
Correct Profile.	Minimum Profile.	Maximum Profile.		Web.	Flange.	Correct Profile.	Minimum Profile.	Maximum Profile.		Web.	Flange.
Inches.	Inches.	Inches.	Lbs.	Inch.	Inch.	Inches.	Inches.	Inches.	Lbs.	Inch.	Inch.
3¼ × 1⅜	...	...	6¾	3/8	3/8	**8 × 3½**	**8 × 3½**	...	22¾	7/16 *b*	1/2
...	3¼ × 1 5/16	...	6	5/16	3/8	...	...	8 × 3 11/16	28	5/8	1/2
...	...	3¼ × 1 7/16	7½	7/16	3/8	9 × 3	...	...	18¼	3/8	3/8
3½ × 1½	...	...	8¼	3/8	1/2	...	9 × 2 15/16	...	16½	5/16	3/8
...	3½ × 1 7/16	...	7½	5/16	1/2	...	...	9 × 3⅛	22	1/2	3/8
...	...	3½ × 1 9/16	9	7/16	1/2	**9 × 3½**	**9 × 3½**	...	22¼	3/8	1/2
3½ × 2	...	...	6¾	1/4	5/16	...	...	9 × 3¾	30	5/8	1/2
...	3½ × 1 15/16	...	6⅛	3/16	5/16	10 × 3½	...	...	27¼	1/2	1/2
...	...	3½ × 2⅛	8¼	3/8	5/16	...	10 × 3⅜	...	23¼	3/8	1/2
5 × 2½	...	...	11	5/16	3/8	...	...	10 × 3⅝	31½	5/8	1/2
...	5 × 2 7/16	...	10	1/4	3/8	12 × 3	...	...	29	1/2	1/2
...	...	5 × 2⅝	13¼	7/16	3/8	...	12 × 2 15/16	...	26½	7/16	1/2
6 × 3	**6 × 3**	...	16¼	3/8	1/2	...	...	12 × 3⅛	34	5/8	1/2
...	...	6 × 3¼	21¼	5/8	1/2	12 × 3½	...	...	30¾	1/2	1/2
6 × 3½	**9 × 3½**	...	18	3/8	1/2	...	12 × 3 7/16	...	28	7/16	1/2
...	...	6 × 3¾	23	5/8	1/2	...	...	12 × 3⅝	35¾	5/8	1/2
7 × 3	**7 × 3**	...	17½	3/8	1/2	**12 × 3½**	**12 × 3½**	...	33	1/2	5/8
...	...	7 × 3¼	23½	5/8	1/2	...	...	12 × 3¾	43	3/4	5/8
7 × 3½	...	...	20¼	3/8	1/2	**15 × 4**	...	...	42	1/2	5/8
...	7 × 3½ *b*	...	19½	3/8	1/2	...	15 × 4 *b*	×	40¾	1/2	5/8
...	...	7 × 3¾	25½	5/8	1/2	...	...	15 × 4¼ *b*	53½	3/4	5/8

Extras—Sizes, under 6 inches wide and over 10 inches wide; length, under 5 and over 40 feet.
Note.—Where the size of the channel is printed in heavy type the section is British standard.

CHAPTER VIII

PILLARS

General Principles.—The values given in the table at the end of Chapter IV. for the ultimate resistance of iron in compression are found by experimenting upon test pieces whose lengths are, say, from $1\frac{1}{2}$ to 3 times their diameters. With pieces of this length, and even up to a length of 6 or 8 times the diameter, the stresses set up, to all intents and purposes, are purely and simply compressional.

If a rod, 40 diameters in length or longer, be put under compression it will immediately begin to bend,

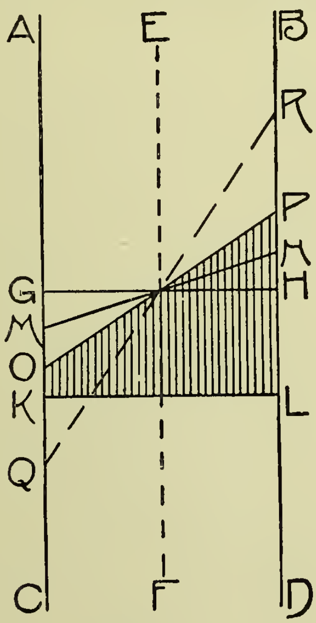


Fig. 93.

and its ultimate strength will depend entirely upon its resistance to bending, and consequently it will bear only a small fraction of the load which would be borne by a short piece of the same sectional area. Between the extreme cases mentioned above, members in compression will fail from the combined effect of direct compression and of bending.

ABCD (Fig. 93) represents the elevation of part of a pillar having EF as its neutral axis. The distance between the lines GH and KL represents diagrammatically a uniform compressive stress per square inch due to a perfectly central load on the top of the pillar. Now if the modulus of elasticity varies on the two sides of the pillar, one side will be more compressed than the other, and the pillar will be slightly bent, producing a bending moment $W \times d$, where W is the load borne by the pillar and d is the extent of the deflection at the centre. This bending moment will produce compression on one side of the pillar equal to HN, and tension GM on the other side. The total compression on opposite sides is now equal to NL and MK respectively. If now the load comes upon the pillar at a distance of x inches from its central axis, another bending moment is introduced equal to $W.x$, producing compression NP and tension MO. The total compression on opposite sides now equals PL and OK. If the stress due to bending is greater than the direct compressional stress there will be tension on one side, as indicated at KQ, while the maximum compressive stress = RL. These considerations govern the strength of all pillars; for no pillar is of perfectly homogeneous structure throughout, and no loading can be perfectly central.

The ultimate strength of the pillar is reached when the stress PL becomes equal to the elastic limit of the material; for if taken beyond this point the deflection, and therefore the bending moment, will be increased, and the maximum stress will reach the breaking-point without any further addition of load.

The theory involved has been investigated by many scientists, but as the necessary calculations are intricate, and as the results arrived at are often no more reliable than purely empirical formulæ, on account of unavoidable irregularities of manufacture and fixing, it would be out of place to enter fully upon them here.[1]

It is evident that the load per square inch that a pillar will support depends approximately upon the ratio of length to diameter. For pillars up to 8 diameters in height the full safe load given in the table at the end of Chapter IV. may be allowed, but above this pillar formulæ must be employed to arrive at it. The use of pillars of greater height than 30 diameters is generally to be avoided.

Radius of Gyration.—The strength of a pillar, as stated above, depends largely upon its resistance to bending, and, as in the case of the beam, resistance to bending depends upon the moment of inertia of the section employed; for moment of resistance $= \frac{I}{y}f$, as explained in Chapter II.

[1] For more information on the theory of this subject the reader is referred to T. Claxton Fidler on "Bridge Construction," and to Johnson, Bryan, and Turneaure on "Modern Framed Structures."

In Fig. 94, suppose the metal on either side of the neutral axis to be disposed in two layers at a distance r on either side of the neutral axis. If the total area of the section = A, the area of each layer $=\frac{A}{2}$. Suppose the two layers to be so thin that the intensity of stress in them is constant throughout. If the stress intensity

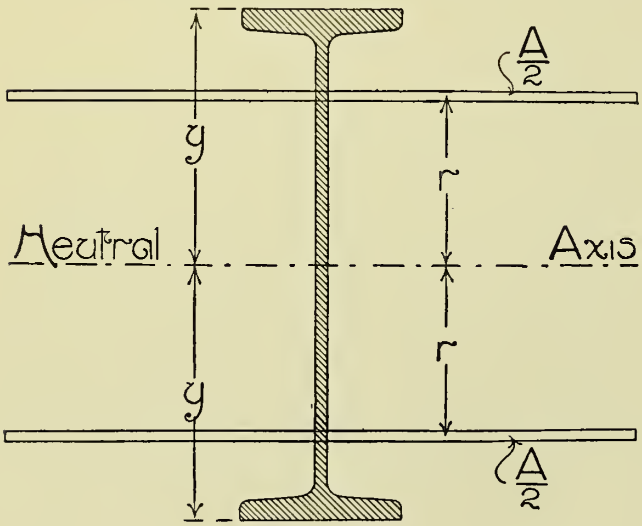


FIG. 94.

in the outer layer of the section be equal to f, the stress intensity at the imaginary layer $=\frac{fr}{y}$, and the moment of resistance of these layers $=2\times\frac{A}{2}\times\frac{fr}{y}\times r$ $=\frac{Afr^2}{y}$.

Now, the distance r may be such that the moment

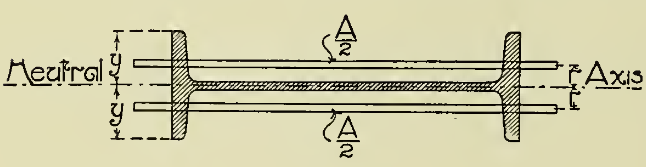


FIG. 95.

of resistance of the layers = the moment of resistance of the actual section, that is to say—

$$\frac{Afr^2}{y}=\frac{I}{y}f.$$

$$\therefore\ Ar^2=I.\quad \therefore\ r^2=\frac{I}{A},\ \text{or}\ r=\sqrt{\frac{I}{A}}.$$

The distance r is known as the Radius of Gyration. Thus the resistance to bending varies as r^2, and the strength of a pillar depends upon the ratio of length to radius of gyration, and not upon ratio of length to diameter.

In considering Fig. 94 it was assumed that bending would take place in a direction parallel to the web, but actually it would evidently take place in a direction at right angles to this. The radius of gyration for the latter case is illustrated in Fig. 95, and is seen to be considerably less than in Fig. 94. Thus "Radius of gyration" in pillar formulæ must always be understood to mean "Least radius of gyration," unless the pillar is prevented from bending in this direction by some other cause.

For a rectangular section $I=\frac{bd^3}{12}$, and $A=b.d.$

$$\therefore\ r^2=\frac{bd^3}{12}\div b.d=\frac{d^2}{12}.$$

$$\therefore\ r=\frac{d}{3{\cdot}45}.$$

But d must be understood to mean the least dimension of the section. The square of the radius of gyration for other sections may be found in the same way, and some are given in Fig. 96. The radius of gyration together with other properties of the Engineering Standards Committee's sections are given in a publication entitled *The Properties of British Standard Sections.*

THE CONDITION OF THE ENDS OF PILLARS.—If a pillar be pivoted at its ends, on being loaded it will bend as shown in Fig. 97. If again, the same pillar be thoroughly fixed at its ends, as indicated in Fig. 98, it will bend as there shown, having two points of contra-flexure. The case is very similar to that of a beam with fixed ends. The length of pillar between the points of contra-flexure, which $=\frac{l}{2}$, may be considered as a pillar with pivoted ends. Thus the resistance to bending in this case = the resistance to bending of a pillar with pivoted ends and of half the length. The bending moment may be proved to vary directly as the square of the length; thus the pillar shown in Fig. 98 is four times as strong as the pillar shown in Fig. 97,—that is, as far as bending is concerned.

In Fig. 99 a pillar is shown with one end pivoted and the other end fixed. The point of contra-flexure occurs at a point two-thirds of the length from the pivoted end, and the resistance to bending is equal to the resistance to bending of a pillar with both ends pivoted and of two-thirds the length. Thus the resistance to bending $=\left(\frac{3}{2}\right)^2=\frac{9}{4}=2.25$ times the resistance of the pillar in Fig. 97.

Similarly the resistance to bending of the pillar in Fig. 100, which has one end fixed and the other end perfectly free, is equal to the resistance of a pillar of twice its length with pivoted ends, and has therefore $\frac{1}{2^2}=\frac{1}{4}$ of the resistance of the pillar in Fig. 97.

The truth of the above statements depends upon the perfection of the pivoting for fixing of the ends; but neither of these conditions is perfectly attained in practice. A "pin-connected" strut is looked upon as

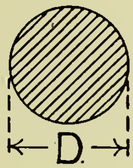
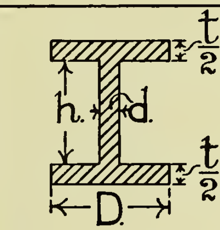
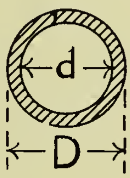
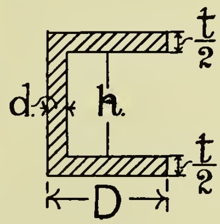
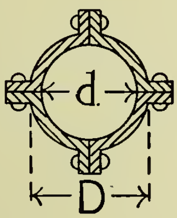
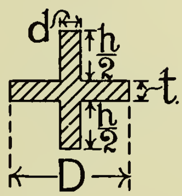
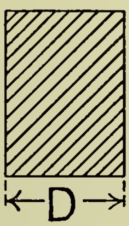
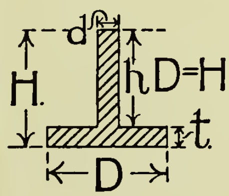
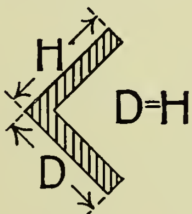
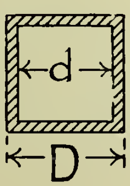
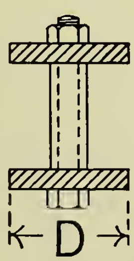
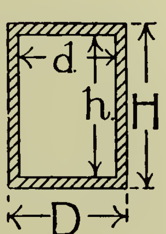
Form of Cross Section.	Square of Least Radius of Gyration. r^2	Approximate Value of r.	Form of Cross Section.	Square of Least Radius of Gyration. r^2	Approximate Value of r.
	$\frac{D^2}{16}$	·25D		$\frac{D^3t+d^3h}{12(Dt+dh)}$	·21D
	$\frac{D^2+d^2}{16}$	·32D		Dº	·28D
	—	·36D		Dº	·20D
	$\frac{D^2}{12}$	·29D		Dº	·21D
				—	·20D
	$\frac{D^2+d^2}{12}$	·37D		$\frac{D^2}{12}$	·29D
	$\frac{D^3H-d^3h}{12(DH-dh)}$	·40D		—	·30D

FIG. 96.

a pillar with pivoted ends; but the strength of the strut is perceptibly increased by the friction of the bolt. On the other hand, it is impossible to obtain a perfectly fixed end; but a column or stanchion with large cap and base is generally supposed to have fixed ends.

The length of a pillar should be measured over all, including cap and base.

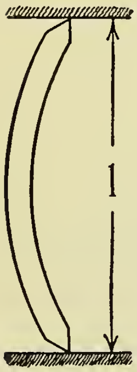


FIG. 97.

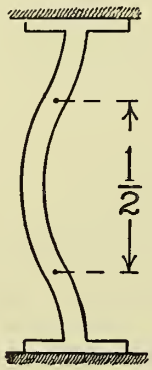


FIG. 98.

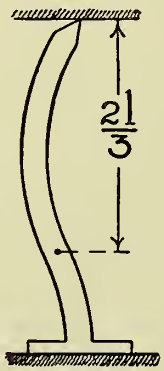


FIG. 99.

It must not be supposed that the strength of pillars with pivoted or fixed ends varies in the proportion given above. Only resistance to bending has been

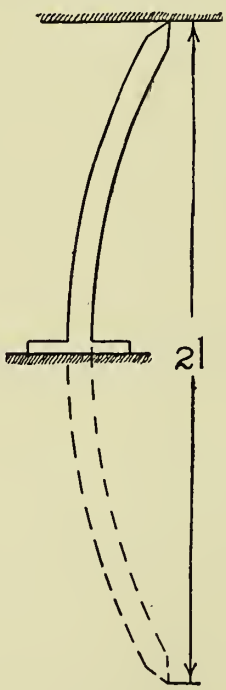


FIG. 100.

there considered; and the actual strength of a pillar depends upon direct compression as well as resistance to bending, except in the case of very long pillars.

CLAXTON FIDLER'S FORMULA.—This is based upon the maximum probable variation in the modulus of elasticity (see commencement of this Chapter).

Ultimate load per square inch in lbs.

$$= \frac{f+p-\sqrt{(f+p)^2-2.4fp}}{1.2}$$

Where—

$p = \pi^2 E \left(\frac{r}{l}\right)^2$ for hinged ends.

$= \pi^2 E \left(\frac{r}{\frac{6}{10}l}\right)^2$ for fixed ends.

r = radius of gyration in inches.

l = length of pillar in inches.

E = modulus of elasticity = 14,000,000 for cast iron.
= 26,000,000 for wrot. iron.
= 29,000,000 for steel.

f = minimum probable ultimate stress in lbs. per square inch.
= 80,000 for cast iron.
= 36,000 for wrought iron.
= 48,000 for mild steel.

This formula is entirely theoretical, and depends upon no experimental co-efficients beyond the value of E and f. It may be relied upon to give safe results. The laborious calculations necessary militate against its general use. (See the table at end of this Chapter.)

GORDON'S FORMULA.—This formula is simple, and is largely used.

For pillars with both ends fixed—

$$\text{Breaking weight in tons} = \frac{fA}{1 + c\left(\frac{l}{d}\right)^2}.$$

For pillars with one end fixed and the other hinged—

$$\text{Breaking weight in tons} = \frac{fA}{1 + 2c\left(\frac{l}{d}\right)^2}.$$

For pillars with both ends hinged—

$$\text{Breaking weight in tons} = \frac{fA}{1 + 3c\left(\frac{l}{d}\right)^2}.$$

f = ultimate resistance of material, tons per square inch.
A = sectional area of metal.
l = length of pillar in inches.
d = diameter or least width of pillar in inches.
c = an experimental constant depending upon the material used and the form of section of the pillar, as given in the table below.

Material.	Section of Pillar.	f	c
Cast Iron.	Solid round . .	36	$\frac{1}{400}$
,,	Hollow round . .	,,	$\frac{1}{800}$
Wrot. Iron or Steel.	Solid round . .	w.i. 16 steel 24	$\frac{1}{2250}$
,,	Hollow round . .	,,	$\frac{1}{3500}$
,,	Solid rectangular .	,,	$\frac{1}{3000}$
,,	Hollow rectangular .	,,	$\frac{1}{5000}$
,,	L T and + sections .	,,	$\frac{1}{1500}$
,,	⊔ and H sections .	,,	$\frac{1}{2000}$
,,	Box sections . .	,,	$\frac{1}{2250}$

The factor $c\left(\frac{l}{d}\right)^2$ in the above three formulæ refers to the strength of the pillar as regards bending, as well as any unavoidable eccentricity of load. Thus, as explained with reference to Figs. 97 to 100, the factor should be approximately $c\left(\frac{l}{d}\right)^2$, $2.25c\left(\frac{l}{d}\right)^2$, and $4c\left(\frac{l}{d}\right)^2$. However, as the absolute fixity of the ends or their perfect freedom to turn upon their hinges is unattainable, the formula as written will give better results. This formula is chiefly useful to arrive at a rough approximation of the strength of a pillar.

RANKINE'S ADAPTATION OF GORDON'S FORMULA.—This formula is practically the same as the last, except that the radius of gyration is used in place of the diameter or width, and it is therefore more trustworthy.

For pillars with both ends fixed—

Breaking weight in tons . . $= \dfrac{fA}{1 + c\left(\frac{l}{r}\right)^2}.$

One end fixed, the other hinged—

Breaking weight . . . . $= \dfrac{fA}{1 + 2c\left(\frac{l}{r}\right)^2}.$

Both ends hinged—

Breaking weight . . . . $= \dfrac{fA}{1 + 3c\left(\frac{l}{r}\right)^2}.$

The significance of the letters is given in the last paragraph.

c is here independent of the form of the pillar, for cast iron $c = \frac{1}{6400}$, and for wrought iron or steel $c = \frac{1}{36000}$.

This formula will give results comparing very favourably with experiment.

JOHNSON'S FORMULA.—The use of this formula is very simple. l and r indicate length and radius of gyration of pillar in inches. w = ultimate load per square inch in lbs.

Cast iron—Fixed ends . . $w = 60{,}000 - 2.25\left(\frac{l}{r}\right)^2.$

Hinged ends . . $w = 60{,}000 - 6.25\left(\frac{l}{r}\right)^2.$

Wrought iron—Fixed ends . $w = 34{,}000 - 0.43\left(\frac{l}{r}\right)^2.$

Hinged ends . . $w = 34{,}000 - 0.67\left(\frac{l}{r}\right)^2.$

Steel—Fixed ends . . . $w = 42{,}000 - 0.62\left(\frac{l}{r}\right)^2.$

Hinged ends . . $w = 42{,}000 - 0.97\left(\frac{l}{r}\right)^2.$

FACTOR OF SAFETY.—All the results given by the formulæ above must be divided by a factor to arrive at the safe load per square inch. Factors of 4, 5, or 6 are commonly used for dead loads. The following formula, due to Mr. Shaler Smith, gives a factor increasing with the length of the pillar to allow for the increased possibility of variations and defects.

Factor of safety $= 4 + 0.05\frac{l}{d}$, which for pillars—

10 diameters long = 4.5
20 ,, = 5
30 ,, = 5.5
40 ,, = 6
50 ,, = 6.5

TABLE.—The following table, which has been compiled from tables given by Prof. Claxton Fidler, and calculated by his formula, has been taken in the main from Longman's *Building Construction*, vol. iv. A factor of safety of 4 has been used throughout.

l = length of pillar, and r = radius of gyration, for value of which see Fig. 96; the approximate values will be found useful with this table.

$\frac{l}{r}$	SAFE STRESS PER SQUARE INCH OF CROSS SECTION.					
	Cast Iron.		Wrought Iron.		Mild Steel.	
	Ends rounded.	Ends fixed.	Ends rounded.	Ends fixed.	Ends rounded.	Ends fixed.
10	8.68	8.85	4.00	4.00	5.33	5.34
15	8.41	8.76	3.98	4.00	5.26	5.31
20	8.07	8.65	3.92	3.99	5.20	5.29
25	7.58	8.46	3.88	3.98	5.13	5.24
30	6.98	8.21	3.80	3.95	5.02	5.20
35	6.32	7.91	3.72	3.92	4.90	5.15
40	5.68	7.56	3.64	3.89	4.76	5.09
45	5.02	7.19	3.54	3.86	4.58	5.03
50	4.43	6.82	3.44	3.82	4.40	4.98
55	3.84	6.46	3.31	3.78	4.22	4.92
60	3.35	6.10	3.17	3.73	4.02	4.83
65	2.92	5.75	3.04	3.68	3.80	4.75
70	2.57	5.39	2.90	3.63	3.59	4.67
75	2.23	5.02	2.76	3.55	3.37	4.56
80	1.96	4.68	2.60	3.48	3.15	4.45
85	1.74	4.33	2.46	3.40	2.96	4.35
90	1.56	4.00	2.33	3.32	2.77	4.25
95	1.42	3.66	2.18	3.22	2.56	4.13
100	1.29	3.35	2.03	3.17	2.40	4.00
105	1.17	3.07	1.92	3.08	2.24	3.88
110	1.07	2.80	1.79	3.00	2.08	3.74
115	0.99	2.57	1.67	2.91	1.95	3.61
120	0.93	2.37	1.57	2.82	1.83	3.46
125	0.86	2.19	1.47	2.74	1.71	3.32
130	0.80	2.03	1.39	2.66	1.61	3.21
135	0.75	1.90	1.32	2.58	1.50	3.09
140	0.70	1.78	1.24	2.48	1.42	2.96
145	0.66	1.66	1.17	2.40	1.36	2.85
150	0.61	1.56	1.10	2.32	1.28	2.72
160	0.56	1.40	0.98	2.14	1.13	2.51
170	0.49	1.25	0.88	2.00	1.01	2.32
180	0.43	1.14	0.80	1.84	0.91	2.13
190	0.39	1.03	0.72	1.70	0.83	1.97
200	0.36	0.93	0.66	1.57	0.75	1.83
210	0.32	0.84	0.58	1.46	0.68	1.68
220	0.30	0.77	0.55	1.35	0.62	1.55
230	0.28	0.70	0.50	1.26	0.58	1.44
240	0.25	0.64	0.46	1.18	0.53	1.34
250	0.23	0.59	0.42	1.11	0.49	1.25
260	0.22	0.56	0.40	1.04	0.46	1.16
270	0.20	0.52	0.37	0.97	0.42	1.08
280	0.19	0.49	0.35	0.91	0.39	1.01
290	0.18	0.46	0.32	0.86	0.37	0.96
300	0.17	0.43	0.30	0.80	0.35	0.92

CHAPTER IX

THE DESIGN OF PILLARS

Rolled steel joists are much used as pillars, either singly or with the addition of plates, as in Fig. 107.

Makers publish tables of safe loads upon their joists used as pillars, and these may be taken as sufficiently accurate for practical purposes. Broad flange beams are particularly suitable for this purpose, as their least radius of gyration is large compared with that of other joists. A table, published by Messrs. Skelton & Co., is given below. It will be noticed that the first ten are square in section, and are therefore very suitable when used singly.

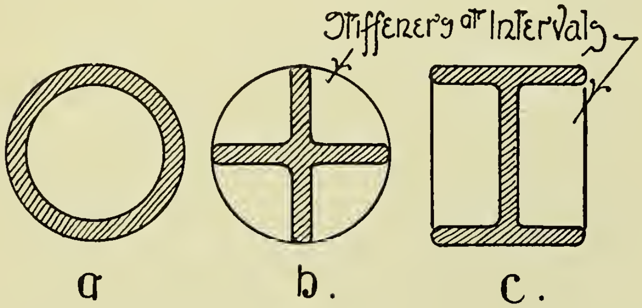


Fig. 101.

Cast-iron Stanchions.—Fig. 101 shows the commonest forms of cast-iron stanchions. All castings should have their angles well rounded as, on cooling, the crystals set themselves in lines at right angles to the surface, as indicated in Fig. 102, producing weak places at sharp angles. This applies chiefly to internal

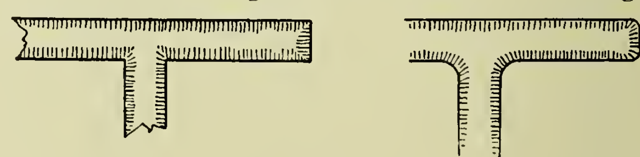


Fig. 102.

BROAD FLANGE BEAMS. TABLE OF PILLARS.

Section No.	Height. Inches.	Width. Inches.	Weight per Foot. Lbs.	Sectional Area. Sq. In.	Moments of Inertia. Axis XX.	Moments of Inertia. Axis YY.	Moments of Resistance (Modulus of Section). Axis XX.	Least Radius of Gyration. Axis YY.	Height of Stanchion. 8′	10′	12′	14′	16′	20′	24′	28′	30′
									Safe Load in Tons.								
18	$7\frac{1}{8}$	$7\frac{1}{8}$	$31\frac{1}{2}$	9.3	84	26	24	1.67	...	46	32	23	18	...	...	...	...
20	$7\frac{7}{8}$	$7\frac{7}{8}$	37	10.91	124	38	32	1.86	...	66	46	34	26	...	...	...	...
22	$8\frac{5}{8}$	$8\frac{5}{8}$	43	12.80	177.298	53.228	40.939	2.040	62	59	55	51	46	36	28	22	20
24	$9\frac{1}{2}$	$9\frac{1}{2}$	51	15.00	246.240	73.032	52.155	2.206	74	70	66	62	58	47	37	29	26
25	10	10	55	16.29	289.584	85.800	58.865	2.295	81	77	74	69	65	53	43	34	31
26	$10\frac{1}{4}$	$10\frac{1}{4}$	61	17.92	344.448	102.264	67.344	2.388	90	86	82	77	73	60	50	40	36
27	$10\frac{5}{8}$	$10\frac{5}{8}$	65	19.10	396.696	118.080	74.664	2.486	96	92	88	84	78	68	55	45	41
28	11	11	69	20.43	457.248	136.104	83.021	2.581	103	99	96	91	86	75	62	51	46
29	$11\frac{1}{2}$	$11\frac{1}{2}$	74	21.87	524.784	154.008	91.988	2.778	111	108	104	101	95	85	72	60	55
30	$11\frac{7}{8}$	$11\frac{7}{8}$	80	23.58	604.824	179.856	102.480	2.761	120	116	112	108	102	91	76	65	59
32	$12\frac{1}{2}$	$11\frac{7}{8}$	85	24.91	722.856	188.808	114.802	2.753	127	123	118	114	108	96	81	68	62
34	$13\frac{1}{2}$	$11\frac{7}{8}$	88	25.94	845.784	194.328	126.543	2.737	132	128	123	119	112	100	83	70	64
36	14	$11\frac{7}{8}$	96	28.15	1019.496	211.032	143.960	2.738	142	139	133	128	121	108	90	75	69
38	15	$11\frac{7}{8}$	101	29.64	1187.804	220.200	158.905	2.725	151	146	140	136	128	113	95	80	73
40	$15\frac{3}{4}$	$11\frac{7}{8}$	107	31.56	1388.016	233.304	176.412	2.718	160	155	149	145	136	121	101	85	77
$42\frac{1}{2}$	$16\frac{3}{4}$	$11\frac{7}{8}$	113	33.16	1637.976	241.872	195.932	2.701	169	163	157	151	142	126	106	88	80
45	$17\frac{3}{4}$	$11\frac{7}{8}$	121	35.55	1941.288	256.032	219.215	2.683	181	175	168	162	152	135	112	94	85
$47\frac{1}{2}$	$18\frac{3}{4}$	$11\frac{7}{8}$	128	37.52	2275.464	267.408	243.512	2.669	190	184	177	169	161	142	117	98	89
50	$19\frac{3}{4}$	$11\frac{7}{8}$	138	40.57	2670.792	281.232	271.511	2.632	206	199	191	184	172	152	126	105	94
55	$21\frac{3}{4}$	$11\frac{7}{8}$	152	44.65	3502.056	301.968	323.666	2.601	226	218	210	200	188	165	137	113	103

The above table is calculated by Euler's formula for pin-ended struts, allowing a factor of safety of 5 to 1. For building work, that is, for ordinary stanchions, the above safe loads can be increased, except for the very short lengths, provided that the ends of the stanchions have the usual amount of fixings.

angles, but also to external angles in a minor degree. To avoid cracks from unequal rates of cooling, any

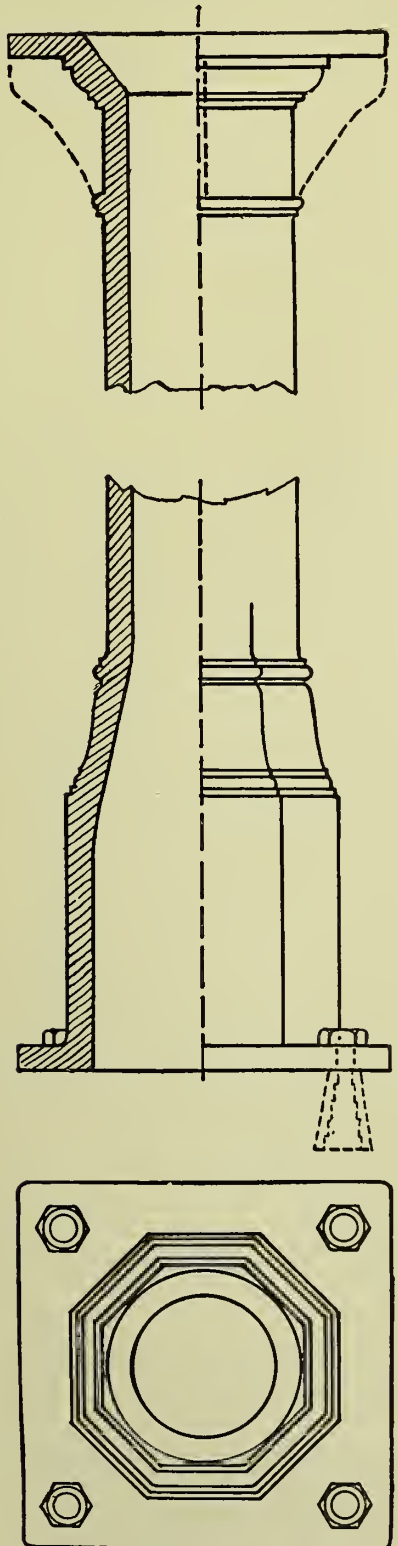


Fig. 103.

increase in the thickness of metal should be as gradual as possible.

The cylindrical section at *a*, Fig. 101, is the most economical of metal, as the latter is all disposed at an equal distance from the centre, besides which there are no angles to weaken it. The sections *b* and *c* have the advantages that flaws and unevenness of thickness are easily detected, and connections are rather more easily formed than in the case of the circular section. To check the thickness of metal in the circular form, small holes must be drilled at intervals.

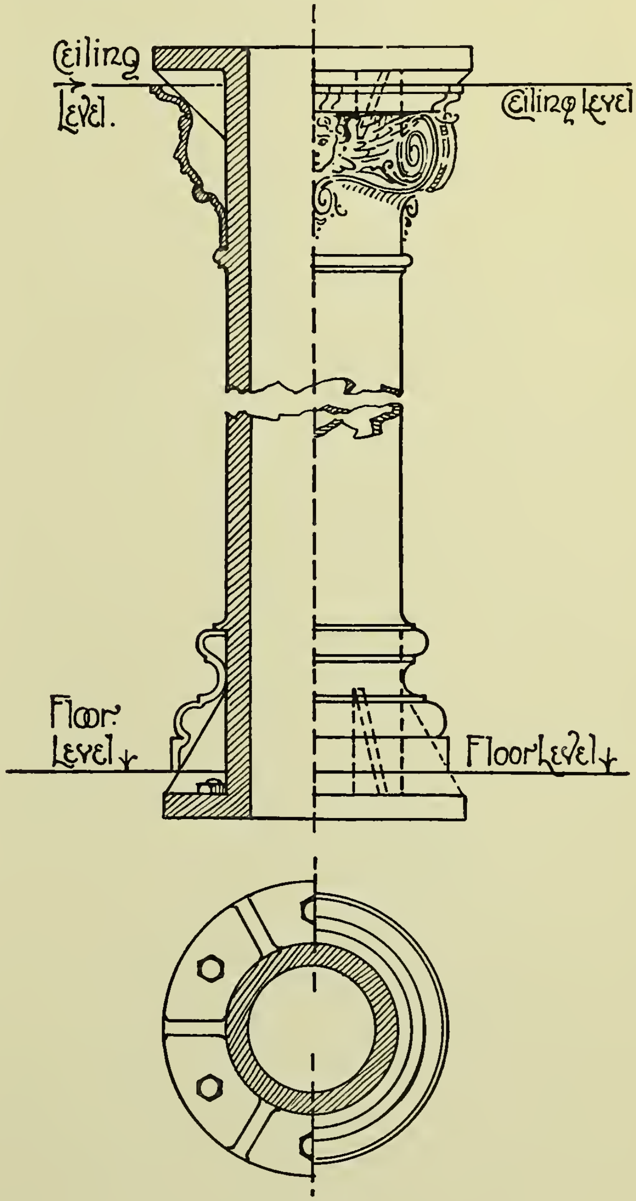


Fig. 104.

Figs. 103, 104, and 105 show the caps, bases, and connections for circular sections, while Fig. 106 shows similar details for an H-shaped section.

Fig. 103 shows a form of column commonly used such as is suitable where the load to be carried is not great. It must be remembered that all mouldings form a source of weakness to the casting, while the increased diameter

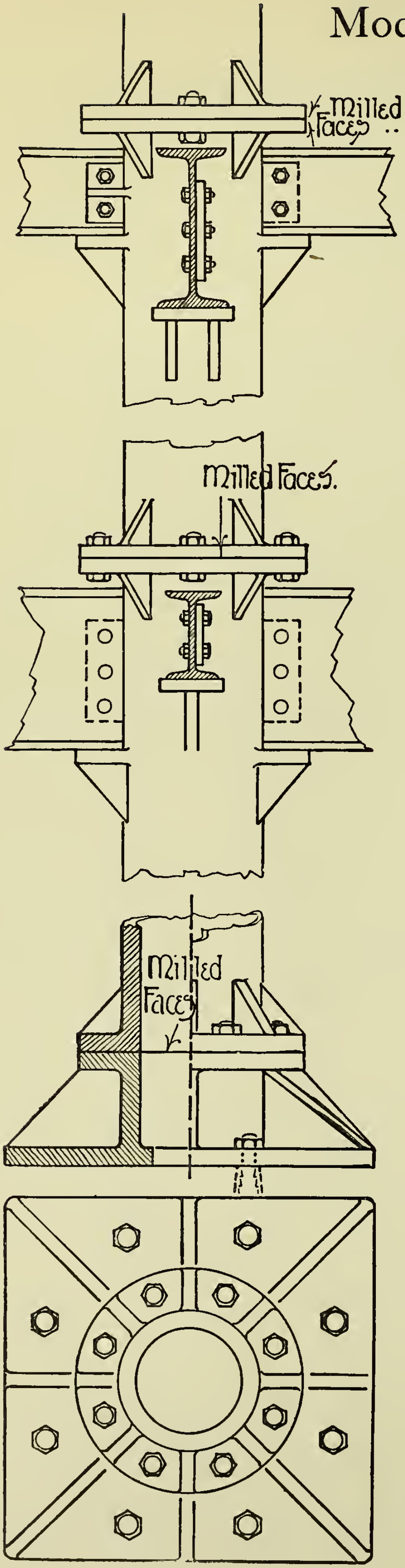


FIG. 105.

at base and cap produce bending strains in the metal at these points. However, if this section be adopted it may be improved by adding feathers in the angle beneath the cap, as indicated in a dotted line. A preferable form is shown in Fig. 104, where the column is plain except for a simple bead, and the ornamentation is added as a separate casting. This ornamentation may be as elaborate as is desired, and is made in two halves fixed together in position.

In certain cases—for instance, when cast-iron columns

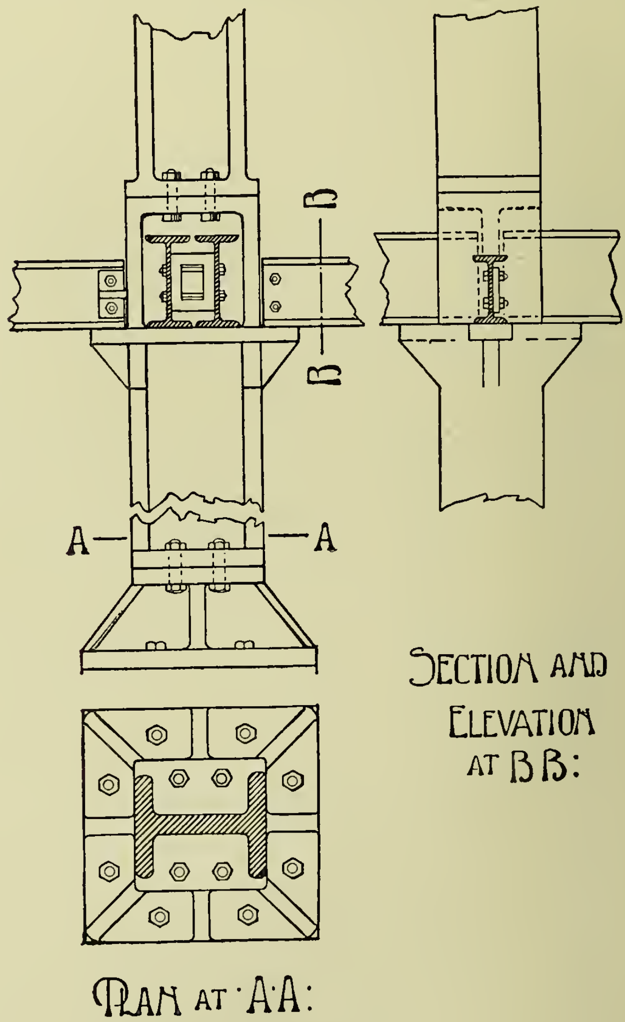


FIG. 106.

are used to support a roof—it is desirable to use a larger diameter of column for the sake of appearance, thus obtaining considerably more metal than is necessary to support the load. Under these conditions greater freedom in ornamentation may be allowed.

When columns are used in positions where they are liable to receive blows their diameter or the thickness of metal must be increased, unless some other means be taken to protect the column.

Figs. 105 and 106 show the joints between two columns one above the other, and also the method of supporting

the ends of rolled steel joists by bolting to lugs cast upon the column.

The same illustrations also show the employment of separate base castings used to spread the load over a larger area of bed stone. Surfaces between column and column, and between column and base plate, should be truly faced.

Cast-iron stanchions are not entirely trustworthy, as they are very liable to cracks and flaws, which make the connections by means of lugs particularly unreliable. These reasons account partly for the fact that steel has largely taken its place for use in stanchions; yet cast iron may often be used advantageously when a single column is required, or when the columns are not carried up over more than one floor. When unprotected from fire, cast iron is more reliable than steel; for, although it may very likely crack in the event of fire, it may continue to support its load, while a stanchion of the latter material will probably entirely give way under similar conditions.

THE DESIGN OF A CAST-IRON COLUMN.—Let it be assumed that it is required to design a cast-iron column 20 feet high to support a dead load of 60 tons.

The diameter of a stanchion should be from $\frac{1}{10}$ to $\frac{1}{30}$ length. First try a diameter of 12 inches, which $=\frac{1}{20}$ length, and assume the ends to be as thoroughly fixed as is practically possible.

According to Gordon's formula (Chapter VIII.), ultimate load in tons per square inch

$$=\frac{f}{1+c\left(\frac{l}{d}\right)^2}=\frac{36}{1+\frac{1}{800}\times(20)^2}=24 \text{ tons}$$

which with a factor of safety of 5 = a safe load of 4.8 tons per square inch.

$\therefore$ Necessary section of metal $=\frac{60}{4.8}=12.5$ square inches, which, with the diameter assumed, would give a thickness of little more than $\frac{1}{3}$ inch.

The thickness of metal should not be less than $\frac{1}{12}$ diameter.

$\therefore$ For economy in metal a smaller diameter, say 9 inches, must be used.

The minimum thickness of metal with this diameter $=\frac{9}{12}=\frac{3}{4}$ inch.

Sectional area of metal $=8\frac{1}{4}\times\pi\times\frac{3}{4}=19\frac{1}{2}$ square inches approximately.

Then by Gordon's formula, total ultimate load

$$=\frac{fA}{1+\frac{1}{800}\left(\frac{l}{d}\right)^2}=\frac{36\times19\frac{1}{2}}{1+\frac{1}{800}\left(\frac{20\times12}{9}\right)^2}=371 \text{ tons.}$$

Using a factor of safety of 5.5—

(*see* p. 91)

$$\text{Safe load}=\frac{371}{5.5}=67 \text{ tons.}$$

In order to illustrate their use, the strength of the column may now be worked out by the other formulæ given in Chapter VIII.

By Rankine's formula, $BW=\dfrac{fA}{1+\frac{1}{6400}\left(\frac{l}{r}\right)^2}$.

$$r^2=\frac{D^2+d^2}{16}=\frac{9^2+7\frac{1}{2}^2}{16}=8.58.$$

$$\therefore BW=\frac{36\times19\frac{1}{2}}{1+\frac{1}{6400}\cdot\frac{(20\times12)^2}{8.58}}=342 \text{ tons.}$$

$$\therefore \text{Safe load}=\frac{342}{5.5}=62 \text{ tons.}$$

By Claxton Fidler's formula, the ultimate load per square inch in lbs. $=\dfrac{f+p-\sqrt{(f+p)^2-2.4fp}}{1.2}$.

$$p=\pi^2E\left(\frac{r}{\frac{6}{10}l}\right)^2=3.1416^2\times14{,}000{,}000\times\frac{8.58}{(\frac{6}{10}\times20\times12)^2}$$
$$=57{,}187.$$

$\therefore$ BW in lbs. per square inch

$$=\frac{80000+57187-\sqrt{(80000+57187)^2-2.4\times80000\times57187}}{1.2}$$
$$=40{,}534 \text{ lbs. per square inch.}$$

$\therefore$ The total safe load for the column, using the same factor as before $=\dfrac{40{,}534\times19\frac{1}{2}}{2240}=64$ tons.

The calculation just given may be saved by the use of the table at the end of Chapter VIII.

$$r=\sqrt{8.58}=2.93.$$

$$\therefore \frac{l}{r}=\frac{20\times12}{2.93}=82.$$

Referring to the table, the safe load is seen to be between 4.68 and 4.33 per square inch, say 4.5.

Total safe load $=19\frac{1}{2}\times4.5=88$ tons.

This result has a factor of safety of 4.

Using the same factor as before, the safe load becomes $\dfrac{88\times4}{5.5}=64$ tons, as above.

By Johnson's formula, ultimate load in lbs. per square inch

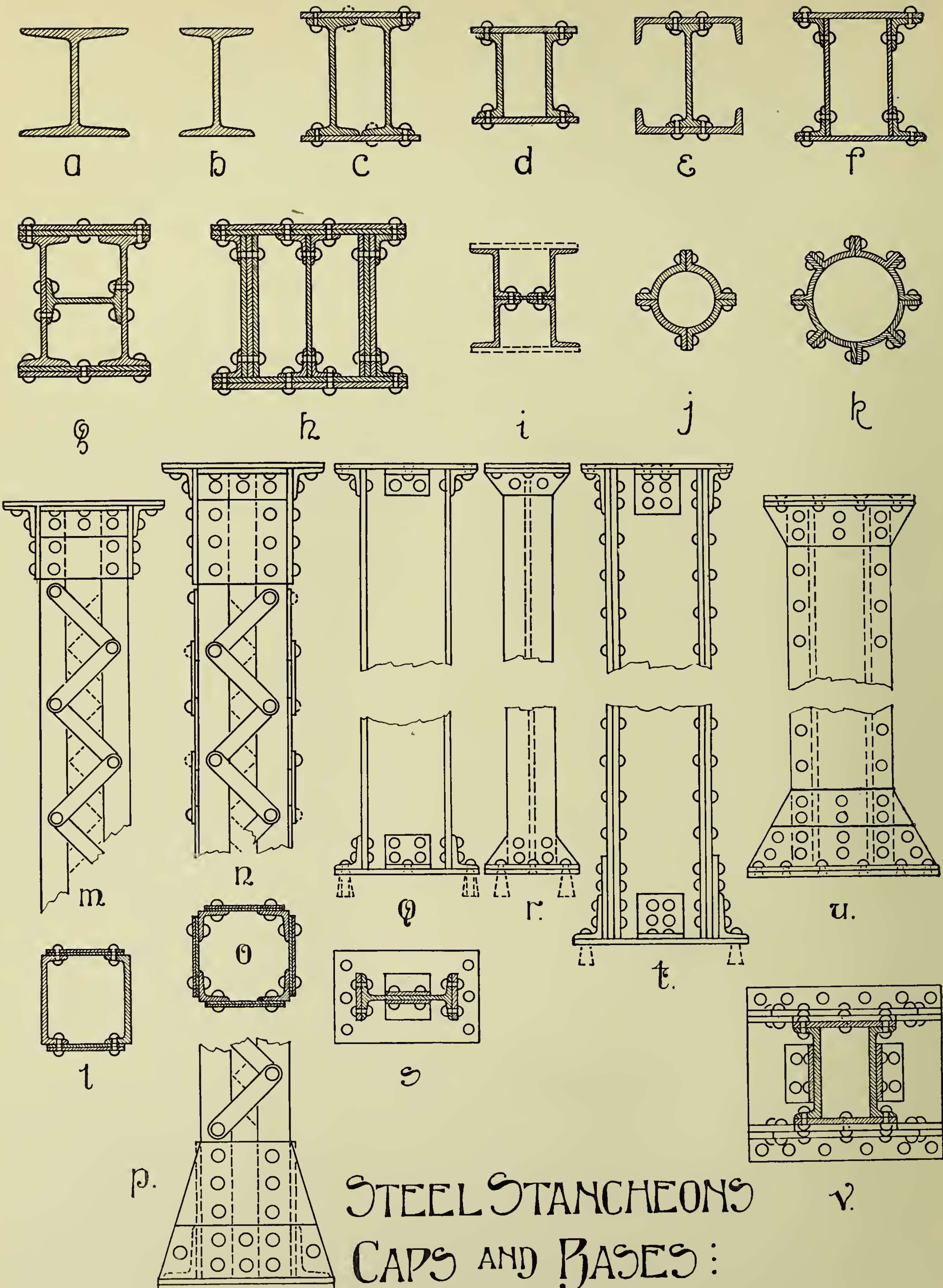
$$=60{,}000-2.25\left(\frac{l}{r}\right)^2$$
$$=60{,}000-2.25\frac{(20\times12)^2}{8.58}$$
$$=44{,}895 \text{ lbs.}$$

$$\therefore \text{Total safe load}=\frac{44{,}895\times19\frac{1}{2}}{2240\times5.5}=71 \text{ tons.}$$

It is therefore evident that the design selected will be strong enough for its purpose.

Allowing a safe load of 15 tons per square foot upon the bed stone, the base plate must have an area of $\frac{60}{15}=4$ square feet.

STEEL STANCHIONS.—The sections of several forms of steel stanchions are shown in Fig. 107. Many other shapes may be used, consisting of various combinations of common sections. Joists, plates, and angles of common sizes may be relied upon for prompt delivery, whereas there may be delay in procuring other sections. Makers will turn rolls to supply any special section to the fancy of the designer if a sufficient quantity be needed, and there be no immediate hurry for delivery.



STEEL STANCHEONS CAPS AND BASES:

FIG. 107.

The aim of the designer should be, so far as it is consistent with rigidity, to place all the metal as far as possible from the central axis, and to use as few rows of rivets as possible.

The section shown at *i*, Fig. 107, has been much used in America, and needs only two rows of rivets; however, it has a rather large percentage of metal at its centre, and when it is necessary to add plates to the section as shown dotted the advantage of few rivets disappears.

At *j* is shown a very economical section as far as amount of metal is concerned, but connections are more difficult than with other forms.

The lattice forms shown at *l*, *m* and *n*, *o*, *p* are particularly useful for supporting light loads. The latticing may generally be formed of metal $\frac{1}{4}$-inch thick,

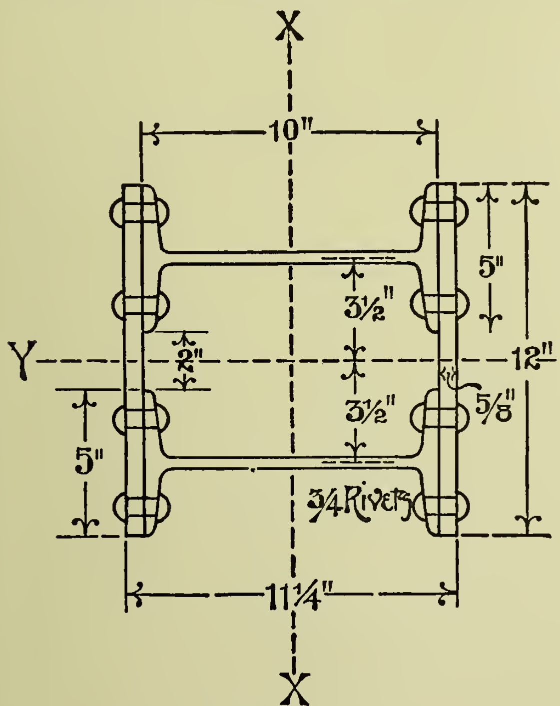


FIG. 108.

and sufficiently wide to take the riveting, say three times the diameter of the rivets.

All sections from *a* to *i* will be improved by the addition of stiffeners at intervals.

From *m* to *v* are shown the method of forming caps and bases with plates, angles, and gusset plates.

THE DESIGN OF A STEEL STANCHION. — Let it be assumed that it is necessary to design a steel stanchion 20 feet high to support a load of 100 tons, and assume that both ends are "hinged" or "rounded" (see Fig. 172).

The selection of a particular section depends partly upon the diameter to be used and the necessary area of metal, and partly upon the fancy of the designer. The reader must not expect to arrive at the most suitable dimensions at once; a system of trial and error must generally be employed.

Let a section be selected here, as shown in Fig. 108, composed of two 10×5-inch joists and two $12 \times \frac{5}{8}$-inch plates.

The area of the section =

2 joists 10×5 inches $= 2 \times 8.82 = 17.64$ (see table in
2 plates $12 \times \frac{5}{8}$ inches . . $= 15$ Chapter V).
∴ Area of section . $= 32.64$ square inches.

By Gordon's formula, ultimate load in tons

$$= \frac{fA}{1 + 3c\left(\frac{l}{d}\right)^2}$$

$$= \frac{24 \times 32.64}{1 + \frac{3}{22500}\left(\frac{20 \times 12}{12}\right)^2}$$

$= 511$ tons.

Slater Smith's factor of safety in this case $= 5$.

∴ Safe load $= \frac{511}{5} = 102$ tons.

The result by this formula should not be relied upon.

The radii of gyration may be found as follows—

About axis YY, $I = 2(I_1 + A_1 h^2) + 2 \cdot \frac{tB^3}{12}$ (see p. 58).

I_1 and A may be found from the manufacturers' tables as in Chapter V. $I_1 = 9.79$ and $A_1 = 8.82$.

∴ About axis YY, $I = 2(9.79 + 8.82 \times 3\tfrac{1}{2}^2) + 2 \cdot \frac{\frac{5}{8} \times 12^3}{12}$.

$= 307.62$.

∴ Square of radius of gyration about axis YY

$$= r^2 = \frac{I}{A} = \frac{307.62}{32.64} = 9.42 .$$

∴ $r = \sqrt{9.42} = 3.07$ inches.

About axis XX, $I = 2I_1 + \frac{B}{12}(D^3 - d^3)$.

In this case $I_1 = 145.6$.

∴ $I = 2 \times 145.6 + \frac{12}{12}(11.25^3 - 10^3)$

$= 715$.

∴ Square of radius of gyration about axis XX

$$= r^2 = \frac{I}{A} = \frac{715}{32.64} = 21.9.$$

∴ $r = \sqrt{21.9} = 4.68$.

Thus the radius of gyration about the axis YY is considerably less than about XX, and only the former case need therefore be considered.

By Rankine's formula, ultimate load

$$= \frac{fA}{1 + 3c\left(\frac{l}{r}\right)^2}$$

$$= \frac{24 \times 32.64}{1 + \frac{3}{36000}\left(\frac{20 \times 12}{3.07}\right)^2}$$

$= 519$ tons.

and safe load $= \frac{519}{5} = 104$ tons.

By Johnson's formula, ultimate load in lbs. per square

$$\text{inch} = 42{,}000 - 0.97\left(\frac{l}{r}\right)^2$$

$= 41{,}940$.

∴ Safe load in tons $= \frac{41{,}940 \times 32.64}{2240 \times 5}$

$= 122$ tons.

By the table in Chapter VIII.—

$$\frac{l}{r}=\frac{20 \times 12}{3.07}=78.18.$$

The safe load may therefore be taken as 3.2 tons per square inch.

$\therefore$ Total safe load $=32.64 \times 3.2=104\frac{1}{2}$ tons.

This result has a factor of safety of only 4, but it is arrived at by Claxton Fidler's formula, with a low value for the ultimate strength of steel.

The section may evidently be considered strong enough for its purpose.

The rivets may be $\frac{3}{4}$-inch diameter, with a 4 or $4\frac{1}{2}$-inch pitch.

Eccentric Loads.—In Chapter VII. it was shown that moment of resistance $=\frac{\mathrm{I}f}{y}=\frac{\mathrm{A}r^2f}{y}$. If a load W be applied to a pillar at distance d from its neutral axis it will exert a bending moment $=\mathrm{W}d$.

$$\therefore \mathrm{W}d=\frac{\mathrm{A}r^2f}{y}.$$

$$\therefore \mathrm{A}=\frac{\mathrm{W}dy}{r^2f}.$$

That is to say, it will be necessary to add an area A to the section of the pillar in order to withstand the bending moment. This is not exact, for the bending moment due to the deflection of the pillar will be slightly increased at the same time, but the above is sufficiently accurate for practical purposes.

In the example of a steel stanchion taken above, suppose 20 tons out of the load of 100 to be applied to the top of the stanchion at a distance of 5 inches from the neutral axis. In the calculations an ultimate stress of 24 tons per square inch has been taken, and a factor of safety of 5 used; therefore it is necessary to make $f=\frac{24}{5}=4.8$ tons per square inch. Then $\mathrm{A}=\frac{\mathrm{W}dy}{r^2f}$ $=\frac{20 \times 5 \times 6}{9.42 \times 4.8}=13$ square inches $=$ the area of metal to be added to resist the bending moment, the value of r remaining constant. However, it will not be necessary to add quite as much as this, as on increasing the sectional area the value of r will also be increased.

It must be understood that the sole object of the additional area of 13 square inches is to resist the bending moment produced by the eccentric load of 20 tons, the dead weight of the load itself being already provided for.

CHAPTER X

FRAMED STRUCTURES

It is evident that the most economical way of supporting a load is by means of a member in direct tension or compression. There is necessarily a quantity of material wasted about the neutral axis of beams, and even of girders which have solid webs. As seen in the last chapter, the conjunction of bending stress and direct compression is most wasteful of all, and should be avoided as far as possible.

In a framework, properly designed, concentrated external forces are met by stresses acting in the direction of the members of the framework ; and their direction being known, their amount can be found by the principles set down in Chapter I., which the beginner should again read at this point.

In Fig. 109, where a weight W is represented as supported by two members hinged together at one

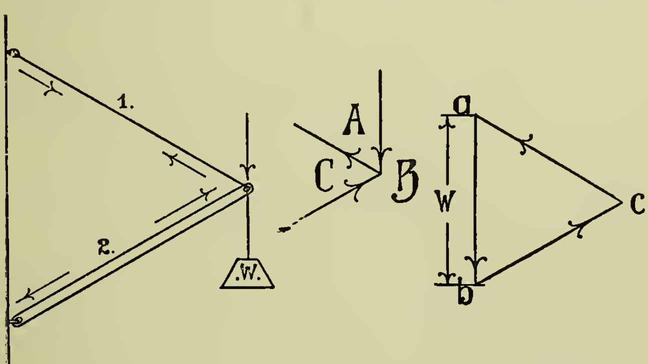


Fig. 109.

end and hinged to a rigid support at the other, it is evident that W will set up a direct compression in one member and tension in the other.

Thus member 1 is in tension and exerts a force acting away from the joints at either end, while 2 is in compression and exerts a force towards the joints. These forces are the components of the downward force of W.

Considering the joint at the junction of the two members, there are three forces in equilibrium acting upon a point as shown in the central sketch ABC, of which BC and CA are the components of AB, and by applying the triangle of forces the amounts of BC and CA are found. Thus in the corresponding triangle of forces *ab* is drawn parallel to AB, representing the amount of W to scale; while *bc* and *ac* are drawn parallel to BC and AC, intersecting at *c* and giving the amounts of the forces in these members. The members about any joint in a stable structure exert forces which are in equilibrium, and therefore the principle of the triangle or polygon of forces can always be introduced.

A Simple Case of Framing.—These principles may now be applied to discover the stresses in a trussed beam, shown diagrammatically in Fig. 110. It must be understood that in all diagrams here given representing the framing of structures the lines represent the central axes of the members, those in compression being denoted by thick lines, and those in tension by thin lines. In practice, however, to ensure accuracy, all lines should be drawn with as fine a line as possible, while the diagrams should be drawn to as large a scale as is convenient. The lettering of spaces has been explained in Chapter I.

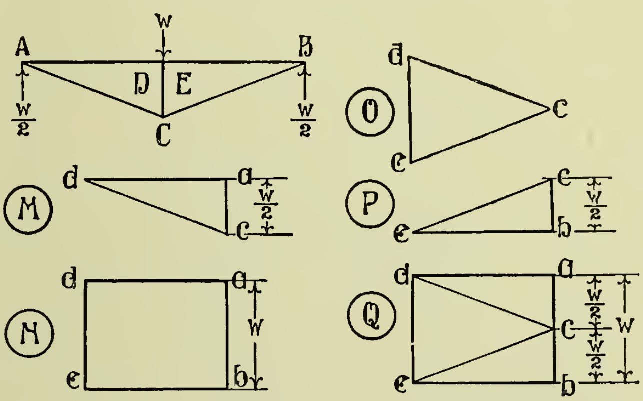


Fig. 110.

First consider the joint at A, which is a case similar to that taken above in Fig. 109, but inverted. By drawing *ca* to represent $\frac{W}{2}$, and by drawing *ad* and *cd* parallel to AD and CD, the stresses in these members are found as shown at M.

Next, considering the joint directly beneath W ; DA being in compression exerts a force towards the joint equal to *da*, as has been already ascertained by M, and this is again set down at N, and *ab* is drawn equal and parallel to W. Then, by the polygon of forces, the stresses in BE and ED are found by drawing *be* and *ed* parallel to them. DE is seen to be in compression to the extent of the weight immediately above it.

In the same way stresses acting at joint C are found at O, *de* being obtained from N. The stresses about the joint at B are shown at P.

It is evident, then, that the stresses round any

joint can be found when not more than two are unknown.

Instead of drawing a separate figure for each joint they may, with greater simplicity and accuracy be combined in one diagram. This is shown at Q, which is a simple combination of M, N, O, and P (Fig. 110).

The Drawing of Force Diagrams.—By the following method nearly all problems in framed structures can be readily solved, the steps taken being illustrated with the case of the double strutted trussed beam shown in Fig. 110.

1st. Letter all spaces between external forces, beginning for preference at the space between the load and reaction as at A. (See also Fig. 111.) F denotes the space between the two reactions. Letter also the spaces within the framing G, H, I. This diagram may be called the "frame" or "space" diagram.

2nd. Set down the loads as they occur to scale upon a "load line," and letter with corresponding letters

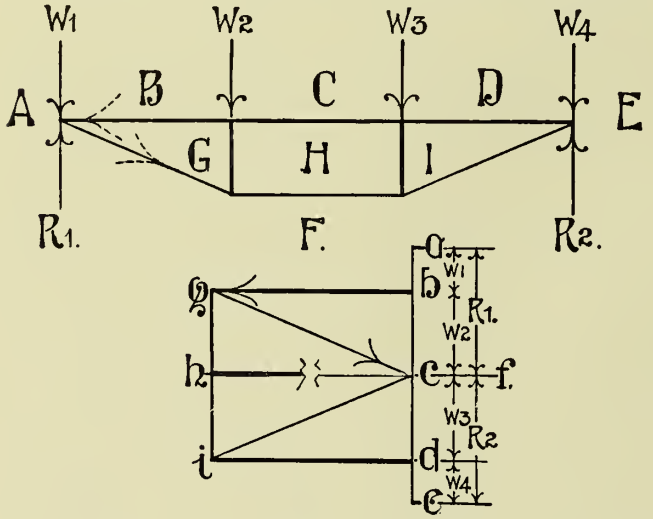


Fig. 111.

a, *b*, *c*, *d*, *e*, the load between the spaces A and B being represented to a scale of weights by the line *ab*, drawn vertically downwards, just as W_1 between A and B acts vertically downwards; and continuing round the diagram in a "clockwise" direction, mark off the reactions *ef*, *fa* upon the load line, arriving back at the starting-point.

3rd. Start preferably at the left-hand joint, or at any joint where there are not more than two unknown forces, and construct triangles or polygons of forces. Thus, starting at the joint at A and passing round the joint in a "clockwise" direction, taking the forces or members as they occur, reaction R_1 has already been set down upon the load line at *fa*. Next, W_1 is reached, which has also been set down at *ab*. Then the forces in BG and GF are found by drawing *bg* and *gf* parallel to them. The polygon for every joint must close at its starting-point.

The force W_1 has no effect upon the framing, and is met directly by the reaction, so that *fb*, or the reaction (less load W_1), may be looked upon as the total external force acting upon the framing at this point.

Next, considering the joint below W_2, the stress in GB and load W_2 are already given on the force diagram by *gb* and *bc*, and the stresses in CH and HG are found by completing the polygon *gbchg*. GB, CH, and HG are all seen to be in compression.

The process is repeated joint by joint until all the members are known.

The direction of the forces is shown on the force diagram by reading off the letters round any particular joint in the order in which they occur on the space diagram, taking care to read off the letters in a "clockwise" direction. Thus, considering the left-hand joint at A, the stress in BG acts in the direction of *bg*, as shown by the arrow, from right to left. On this arrow being transferred to the corresponding line on the space diagram, as shown by a dotted arrow, it is seen to act towards the joint, and the member is therefore in compression. The stress in GF acts in the direction of *gf*, or left to right, and the member is therefore in tension, as the transferred dotted arrow is shown, pulling away from the joint under consideration.

This process also can be repeated joint by joint; or any joint may be considered irrespectively of all others, so long as the clockwise sequence is maintained. Thus at the joint under W_1 this sequence of spaces is ABGFA, and the sequence on the force diagram which determines the direction of the arrows is consequently also *abgfa*. If the joint at the foot of the strut under W_2 be next considered the clockwise sequence of the spaces is GHFG, and the corresponding sequence on the force diagram *ghfg*. It is left for the student to add the arrows, acting from *g* to *h*, *h* to *f*, and *f* to *g*, with the result that the corresponding dotted arrow on the line between G and H acts downwards, towards the joint under consideration, showing that the member is in compression, while the others act away from the joint, denoting tension.

It will then be noted that there are *two* dotted arrows on the line between F and G, but each acts away from the joint which was under consideration when it was applied, and so each shows that the member is in tension; which fact has merely been twice ascertained.

Trussed Beams.—In a properly framed structure no rectangular figure should appear, and any such space should be divided up into triangles. Thus Fig. 111 does not show a true case of framing. In drawing the force diagram, W_2 and W_3 were assumed to be equal; but if this were not the case, or if W_3 were entirely removed, the tie HF, IF would tend to straighten, forcing up the strut HI and causing a bending moment in CH, DI. Therefore a brace must be added within the central rectangle as shown in Fig. 112, in which the force diagram for this arrangement is also shown. Now, even if the loading above the two struts be equal, the equality may always be upset by temporary loads.

in the shape of workmen, etc. Thus this central rectangle should always be braced, and in two directions, as in Fig. 113, to allow for extra accidental loading which may come upon the structure from either end. This does not apply in the case of a trussed timber beam, for here there will always be abundant material

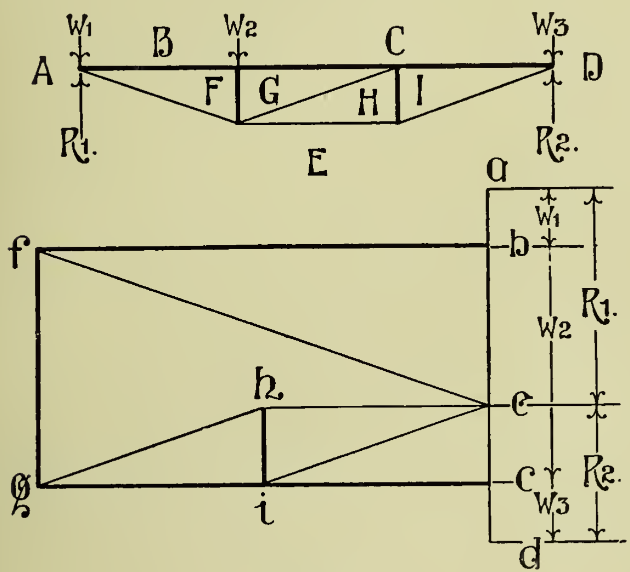


FIG. 112.

to resist this bending moment. If these members HK and KL were of large section, when the loading was even, they would both act as struts, relieving the struts GH and LM of part of their load; but this is not their object, and, being formed merely of light bars, their compressive resistance can be entirely neglected.

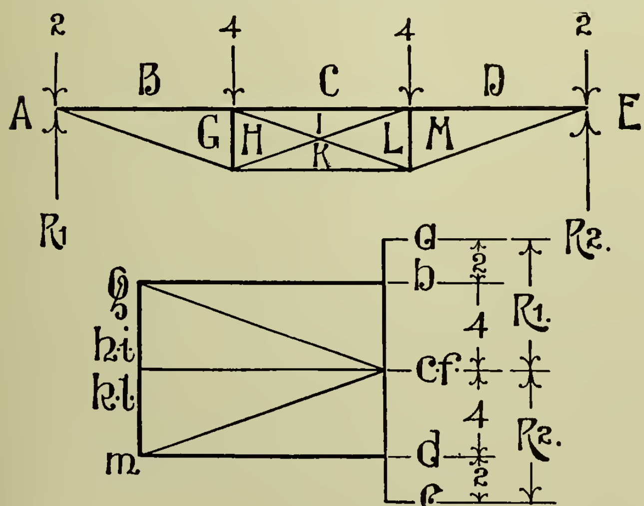


FIG. 113.

Thus the force diagram for this case, as shown in Fig. 113, will be the same as in Fig. 111; but it should be noticed that the four letters *hikl* are found at one point.

Referring again to Fig. 110, it is evident that if the horizontal member be allowed to deflect it will be under a bending stress, and will relieve member DE of a part of its load. This is corrected by tightening the tension rods by means of "turnbuckles," as shown at Fig. 146. If the load along the top of the beam be evenly distributed, the case will be similar to that of a continuous beam over two spans (see Chapter IV.), and the load upon the strut will be equal to $\frac{5}{8}$ of the total load. In this case member AD must be calculated as for a pillar under combined bending stress and direct compression (Chapter IX.). Member DE is calculated as a pillar, while DC and EC are simply proportioned to take their direct tensile stress. Trussed beams of

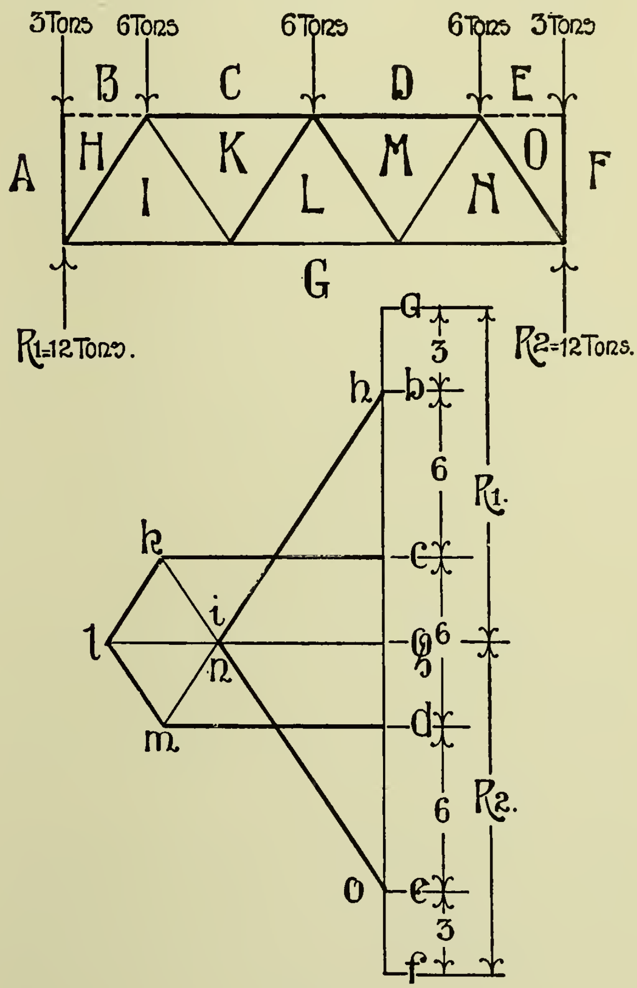


FIG. 114.

this character are largely employed as purlins in roof construction.

LATTICE GIRDERS.—The type of girder described in Chapter VII. is very suitable for supporting walls, on account of its rigidity, but for the main girders of bridges or for girders used to support a number of roof trusses, etc., the following types of framed structure are generally more economical.

WARREN GIRDERS.—Fig. 114 shows a type of girder known as a Warren Girder. It is shown with loads of 3 and 6 tons at the joints along the upper flange or "boom," and the force diagram, which may be drawn by

methods already explained, is given below the framed diagram. Starting at the left-hand end, the joint under AB is seen to have only two unknown members, and on attempting to draw the force diagram for this joint it is found that HA is in compression to the extent of the load immediately above it, while BH is not stressed at all, and has been indicated with a dotted line on the diagram. This result should be self-evident, and the diagram may be started at once at the joint immediately above reaction R_1.

Fig. 115 shows the same girder loaded at the joints of the lower boom. It should be evident that both members 1 and 2 are without stress, for any thrust imparted to member 1 by R_1 can only be met by the resistance to bending of member 2; but member 2 is assumed to have no resistance to bending at its right-hand joint. It is thus seen that these members do not resist any of the reaction, and they may be entirely omitted from the design. Having decided this point,

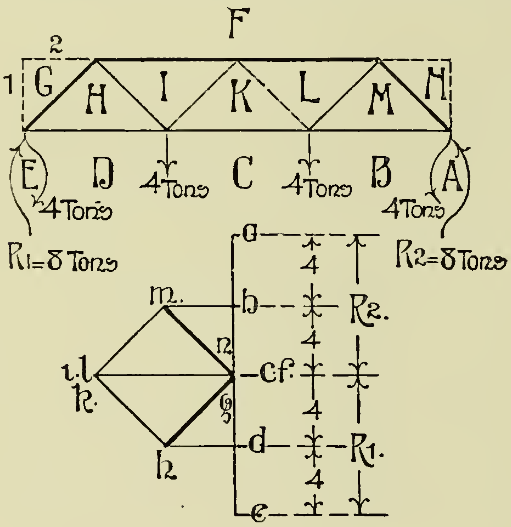


FIG. 115.

the force diagram can be started at the joint immediately above R_1, where there are now only two unknown forces. On drawing the diagram it is found that members IK and KL bear no stress; but this will only be the case when the loading is symmetrical. It will be noticed that the lettering used in this case is slightly varied, and this has been done in order to preserve the clockwise method of lettering; also, the lines indicating the loads and reactions at the ends of the girder have been curved to allow letters to be placed between.

Fig. 116 shows a similar girder in which the position of the diagonal members is reversed. It is obvious that member FE is compressed to the extent of R_1, while FD is devoid of stress. The force diagram is shown as usual below the frame diagram. This case should be compared with that in Fig. 114.

COMPOSITE WARREN GIRDERS.—Fig. 117 shows a form of girder which is a simple combination of the two forms shown in Figs. 115 and 116, and to emphasise this fact the loading assumed in those cases has been adopted here. Each diagonal member is stressed to the same extent as the corresponding member in Figs. 115 and 116, while the booms are stressed to an amount equal to the sum of the stresses in the corresponding parts of the booms in those figures.

On starting to draw the force diagram at the left-hand reaction three unknown members are found, and before the diagram can be proceeded with the stress in the vertical member must be ascertained. To do this the girder must be divided up into its component parts as in Figs. 115 and 116. This can often be done mentally, but in any case it is only necessary to discover what part of R_1 is due to the loading upon that component of the girder whose diagonal comes to the upper end of the vertical strut, and the strut is then

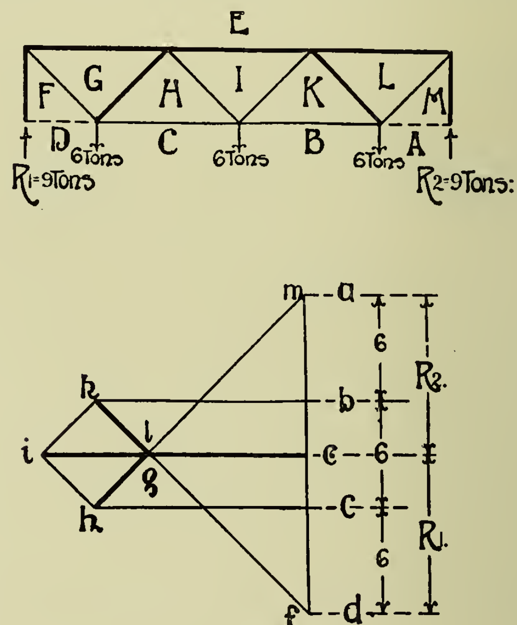


FIG. 116.

known to be in compression to this extent; for, as seen in Fig. 115, the vertical member is without stress in that component of the girder which has its diagonal attached to the lower part of the vertical strut.

It is seen that the stress in each diagonal is shown twice upon the force diagram; for instance, KM and LN represent one member, and likewise with the other diagonal members. By measuring on the force diagram it will be seen that the stresses in LN, NO, RT, TU, etc. are the same as in GH, HI, IK, KL, etc. (Fig. 115), and the stresses in MN, NP, ST, TV, etc. are the same as in FG, GH, HI, IK, etc. (Fig. 116), while the stress in OI = the sum of the stresses in IF (Fig. 115) and GE (Fig. 116), and similarly with other portions of the boom.

If the girder be loaded as in Fig. 118 the force diagram will be precisely the same as that in Fig. 115, and that component of the girder which is similar to Fig. 116 will remain unstressed.

COMPOSITE WARREN GIRDERS WITH VERTICAL

MEMBERS.—By the addition of vertical members in each bay of the girder the two components are tied

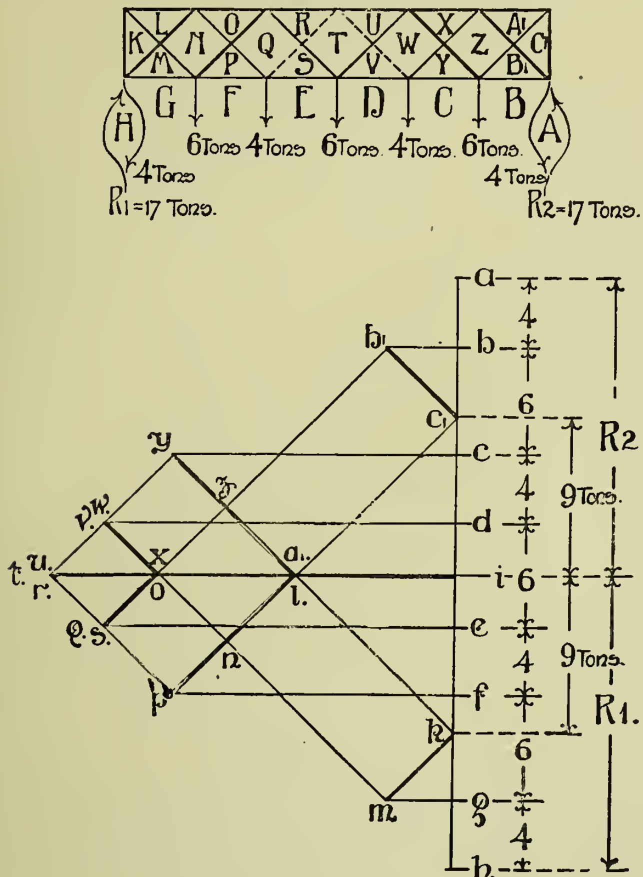


FIG. 117.

together, and the effect of every load is divided equally between them. Thus in Fig. 119 the vertical members

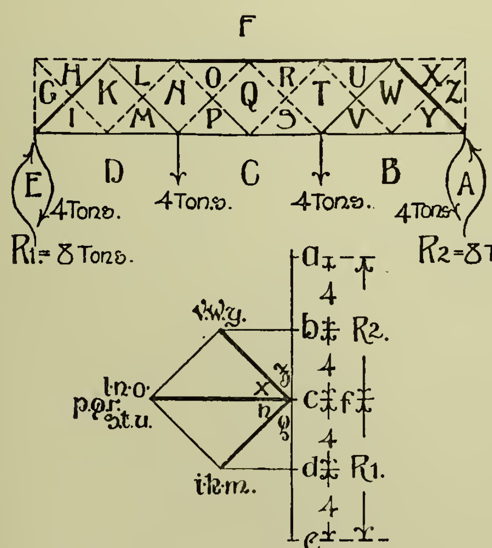


FIG. 118.

immediately over the loads carry half of each of the loads of 4 tons to the upper boom,—that is to say, they are in tension to the extent of two tons. The load may be regarded as being applied directly to the vertical member, and as the framing at either end of it is symmetrical it will evidently share its load between the two components of the girder. The verticals with no load beneath them remain without stress.

Fig. 120 shows a similar girder loaded as in Fig. 117, every load being divided between the framing at the upper and lower booms, and every vertical, except those at the ends, being in tension to the extent of half the load beneath it. The end verticals are in compression to the extent of half the reactions *acting upon the girder*,

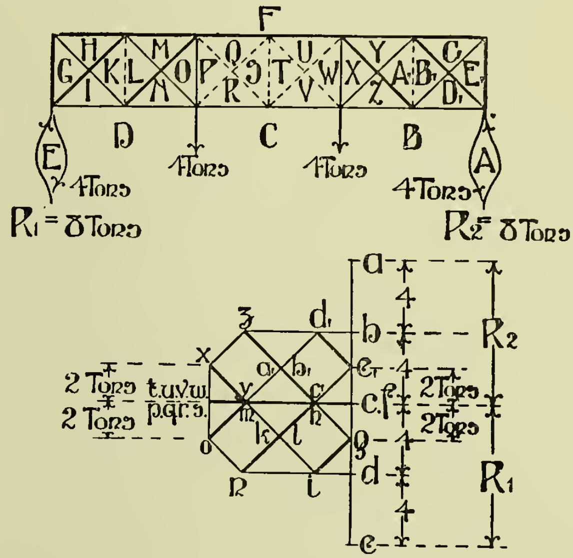


FIG. 119.

it being remembered that the end load of 4 tons is met directly by the reaction and does not affect the strength of the girder. Thus IK is in compression to the extent of $\frac{17-4}{2}=6\frac{1}{2}$ tons. Fig. 120 should be compared with Fig. 117.

Fig. 121 shows a lattice girder of 7 bays with an evenly distributed load along the top boom, besides other loads at the lower boom. To find the reactions, which in this case are unequal, the principles given in Chapter I. are employed. Thus R_1 = half the distributed load and a proportion of the loads along the lower boom $= 28+\frac{6}{7}\times 4+\frac{5}{7}\times 17+\frac{3}{7}\times 8 = 47$ tons. Of this amount, 4 tons is met directly at the head of the strut AO, the remaining 43 tons acting upon the framing of the girder at either end of the strut. Thus the amount of the compression in $AO=\frac{43}{2}+4=25\frac{1}{2}$ tons. The stresses in all vertical members have been figured on the force diagram. No difficulty should be found in drawing this if the method described already be carefully adopted. As the vertical members are met with, the stresses in them must be plotted to scale, having arrived at their amounts as stated above. It should be noticed that, in this case, they are in compression except where the load is greater at the lower boom. Where the

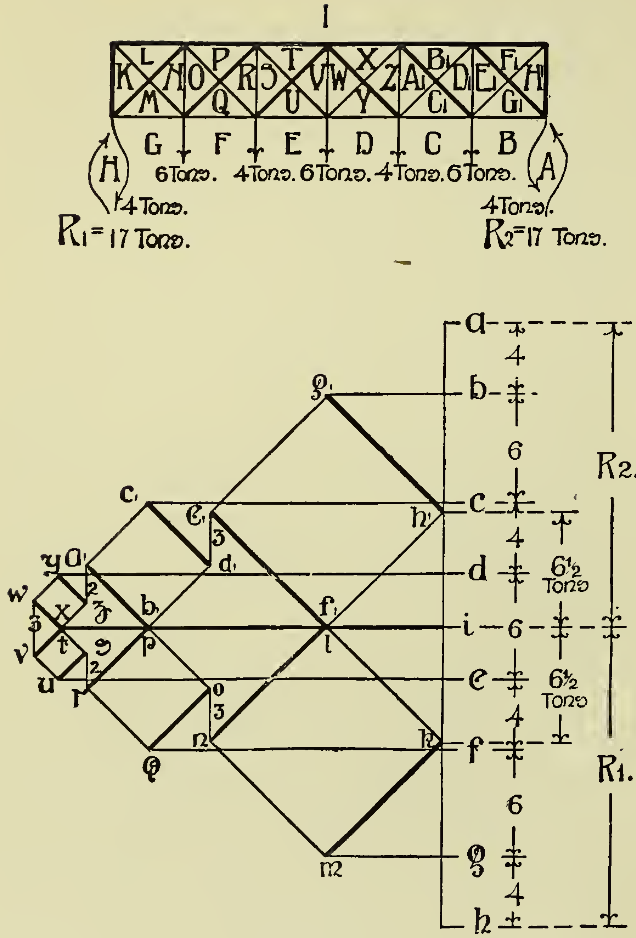


Fig. 120.

vertical is equally loaded at either end the member remains unstressed; or, more correctly speaking, under two mutually counteracting stresses.

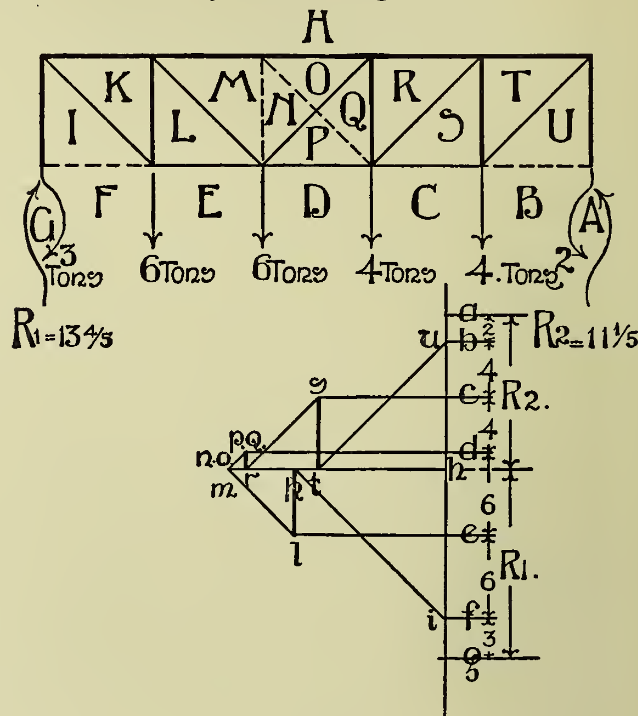


Fig. 122.

Linville Girder.—Fig. 122 shows a form of girder which may be looked upon as an extension of the trussed beam (Fig. 112), and gives both frame and force diagrams, a loading on the lower boom being assumed. It has the

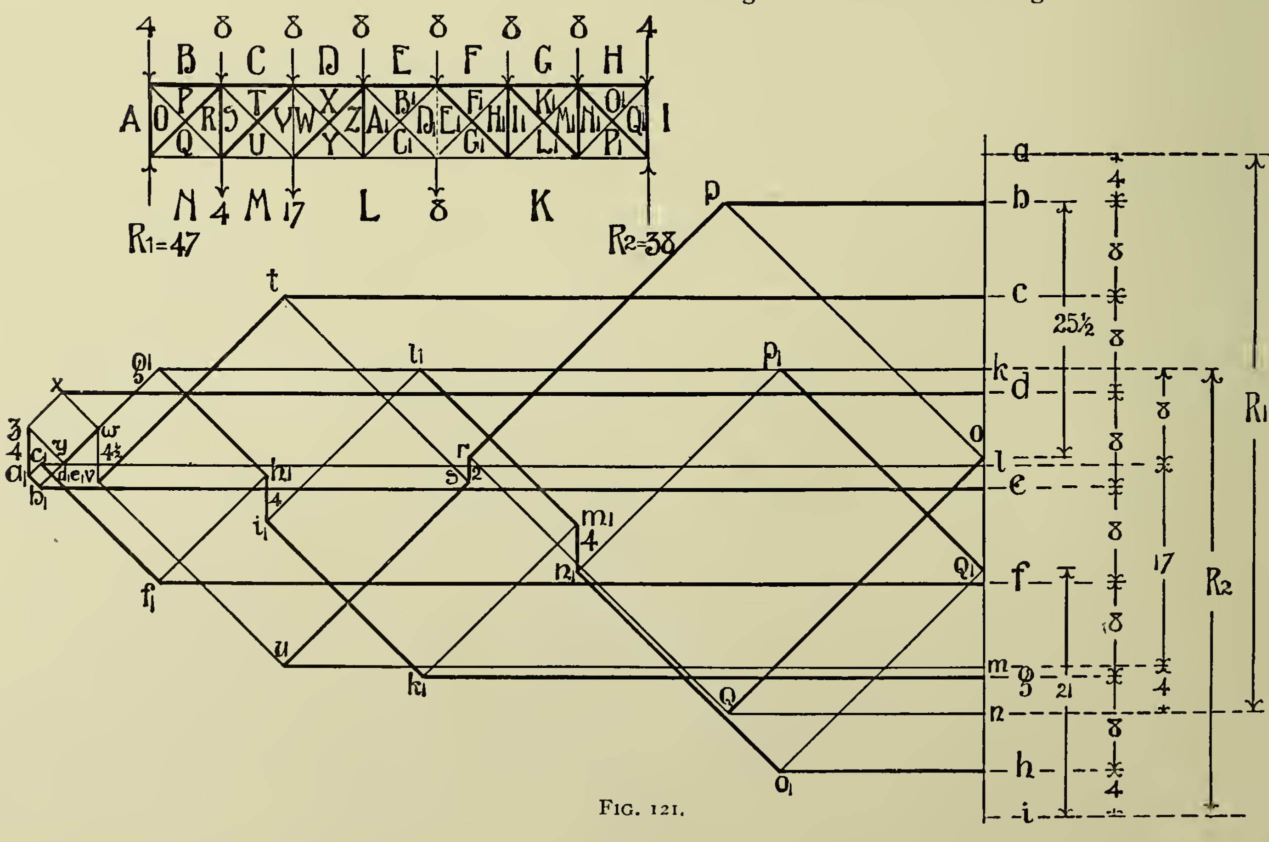


Fig. 121.

advantage that the compressional web members all take the form of vertical struts, and are consequently of minimum length for any given depth of girder. The diagonal members are all designed as ties, and therefore if, in drawing the force diagram, it would appear that any of these are put in compression, their resistance must be entirely neglected and "counter braces" will be necessary, as at the central bay of the girder illustrated. Variations of this form of girder are largely used for heavy bridge work, where the dead weight of the bridge will counteract the necessity for an extended use of counter-bracing, and also to some extent for light girders supporting fixed dead loads; but for light bridges of small span where the live load bears a large proportion to the dead load of the bridge they are not so suitable.

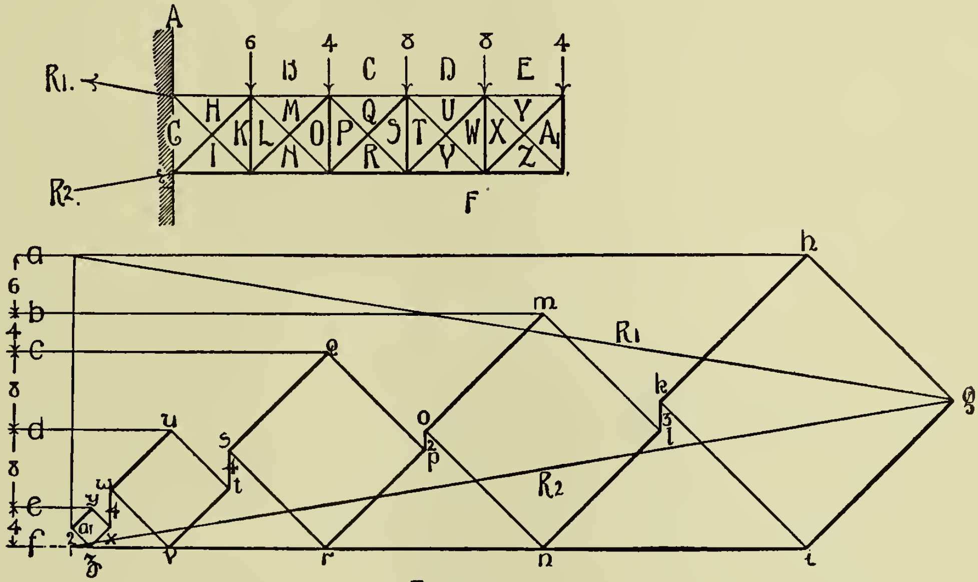


FIG. 123.

DEFLECTION.—The question of deflection is conveniently investigated when dealing with framed girders, and for greater simplicity the case of a cantilever (Fig. 123) may be taken. The force diagram, as given here with an assumed loading, is started at the right-hand end, member FA_1 being in compression to the extent of half the load over it. The reactions, on completing the diagram, are found by joining *ga* and *gf*.

Considering the first bay of the structure (Fig. 124), the web members have very little effect upon the deflection, and so, for practical purposes, may be left out of consideration. Then, if f_1 = intensity of stress in the booms, or the mean intensity if this is not the same in both, and x_1 = length of this portion of the cantilever, the extension of the top flange and the reduction in length of the bottom flange $= \frac{f_1 x_1}{E}$ (see beginning of Chapter IV.).

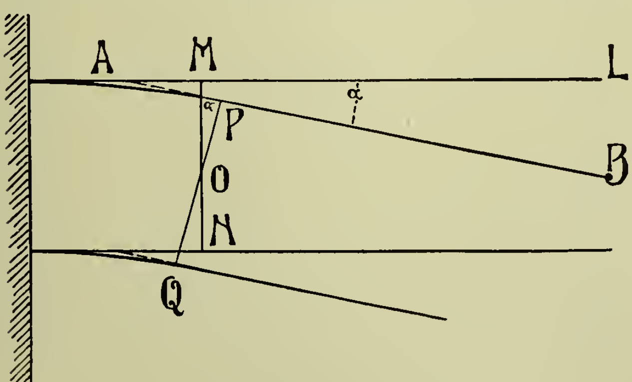


FIG. 124.

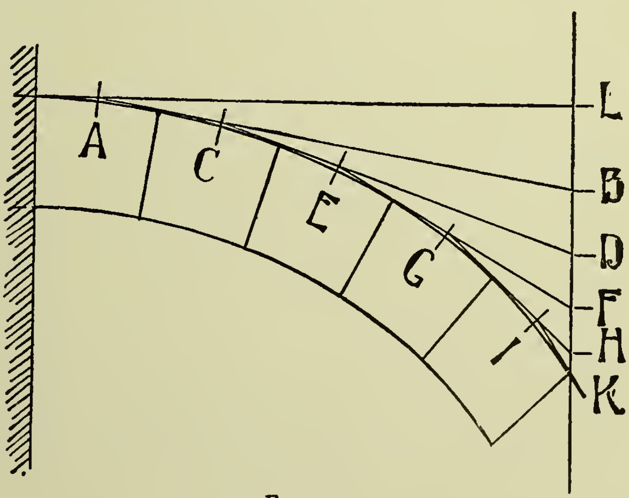


FIG. 125.

The bay is thus distorted as shown, in an exaggerated form, by the fine lines, and the rest of the cantilever will have a slope due to this distortion, the direction of slope being given by the tangent AB to the curve at point P. The vertical member MN takes position PQ, and as MN and PQ are respectively perpendicular to AL and AB, $\angle$ LAB = $\angle$ MOP = α.

Then $\tan \alpha =$ extension or compression of flange $\div$ OP $= \frac{f_1 x_1}{Ey}$ where $y = \frac{d}{2} =$ OP.

Therefore the deflection LB at the end of the beam (Fig. 125), caused by the distortion of the first bay, $= \text{AL} \tan \alpha = \text{AL} . \frac{f_1 x_1}{Ey}$.

The deflection BD due to the distortion of the second bay similarly $= \text{CB} . \frac{f_2 x_2}{Ey}$, where $f_2 =$ intensity of stress in the flanges of the second bay, and $x_2 =$ the length of these flanges.

Thus the total deflection at the end of the cantilever $= \text{AL} . \frac{f_1 x_1}{Ey} + \text{CB} . \frac{f_2 x_2}{Ey} + \text{ED} . \frac{f_3 x_3}{Ey} + \text{etc.}$

Now if the cantilever be so proportioned that f and x are constant in every bay the deflection becomes $\frac{fx}{Ey}$ (AL + CB + ED + etc.).

It may be shown that $x(\text{AL} + \text{CB} + \text{ED} + \text{etc.}) = \frac{l^2}{2}$.

$\therefore$ Total deflection $+ \frac{1}{2} \frac{fl^2}{Ey}$; which is the formula given for this case in Chapter IV.

CHAPTER XI

THE DESIGN OF LATTICE GIRDERS

A Design for a Lattice Girder.—Fig. 127 is a design for a lattice girder to support the extremities of four roof trusses spaced 12 feet apart. The trusses are supported in pairs, and the girder thus spans three bays of the roof.

The total weight of each roof truss, together with the load that it may be called upon to support, has been assumed to be 10 tons, half of which amount is brought upon the girder by each truss. Thus $5 \times 2 = 10$ tons is the load at each of the two points of support of the trusses.

The type of girder selected is the double Warren form shown in Fig. 117, with the addition of vertical struts under the loads. Were it not for the latter members, one system of web members would alone be affected; but their addition divides each load equally between the component systems, as explained in the last chapter.

Only half the stress diagram has been given, as the other half will obviously be a repetition of that shown.

Flanges.—The maximum stress in the flanges is found by the diagram to be 30 tons. Allowing a safe tensional stress of $6\frac{1}{2}$ tons per square inch, the lower flange will need a sectional area of 4.61 inches. Several methods of constructing the flanges of lattice girders are shown in Fig. 126. Considering the form of the joints to be adopted between the members, a flange consisting of two angles $5 \times 2\frac{1}{2} \times \frac{3}{8}$ inches may be tentatively selected, which, allowing for two $\frac{5}{8}$-inch rivet holes, have a total sectional area of 4.41 inches. This is slightly less than 4.61, but may be safely employed.

Now, considering the compression flange, its length between successive joints = 47 inches. According to the table in Chapter VIII., the approximate value of $r = .2D$, and for the section adopted $r = .2 \times 5 = 1$ inch. $\therefore \frac{l}{r} = 47$. By the subsequent table at the end of Chapter VIII., considering each portion of the flange as having hinged ends, safe stress per square inch = 4.5 tons. Sectional area of angles = 5.34 square inches (rivet holes not being deducted in compression members). Then safe load = $5.34 \times 4.5 = 24$ tons. For members AF and AI this is ample, but at the centre of the girder the flange must be strengthened, and this has been effected by riveting a $6 \times \frac{5}{16}$ inches flat to its upper side, giving a total sectional area of 7.2 square inches, and a safe load of 32.4 tons.

In order that "r" in a horizontal direction may be as great as "r" measured vertically, the two angles

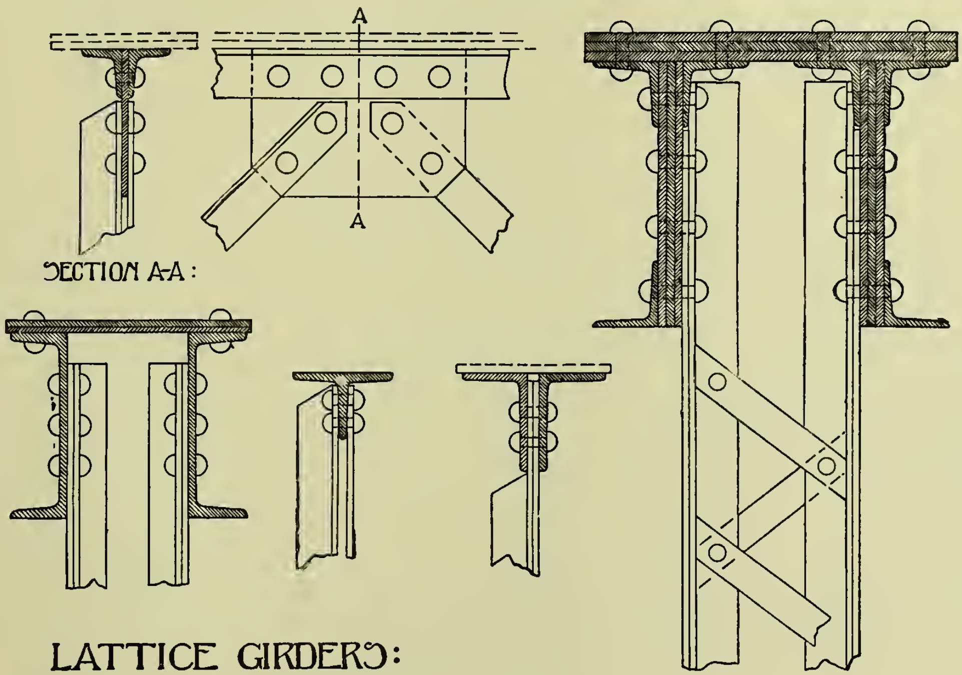


Fig. 126.

must be riveted together between the joints, the rivets passing through distance pieces to keep the angles properly separated.

The upper flange has been kneed at its extremities, and has been brought down to form the "end pillars," being joined to the lower flange by means of cover plates.

Web Members.—The stress diagram shows that the struts and braces between load and reaction are uniformly stressed to the extent of 7.06 tons, while of the struts, and in fact the effective length of these might be considered as being half their total length; but the safety of this assumption will depend upon the existence of a single rivet at the crossing, as well as upon the absence of any external tendency to bending; besides which, by strengthening the struts, the rigidity of the whole structure will be increased.

Considering the effective length of the struts as half their total length, angles $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}$ inches would be found sufficient; or, taking it as the total length, angles

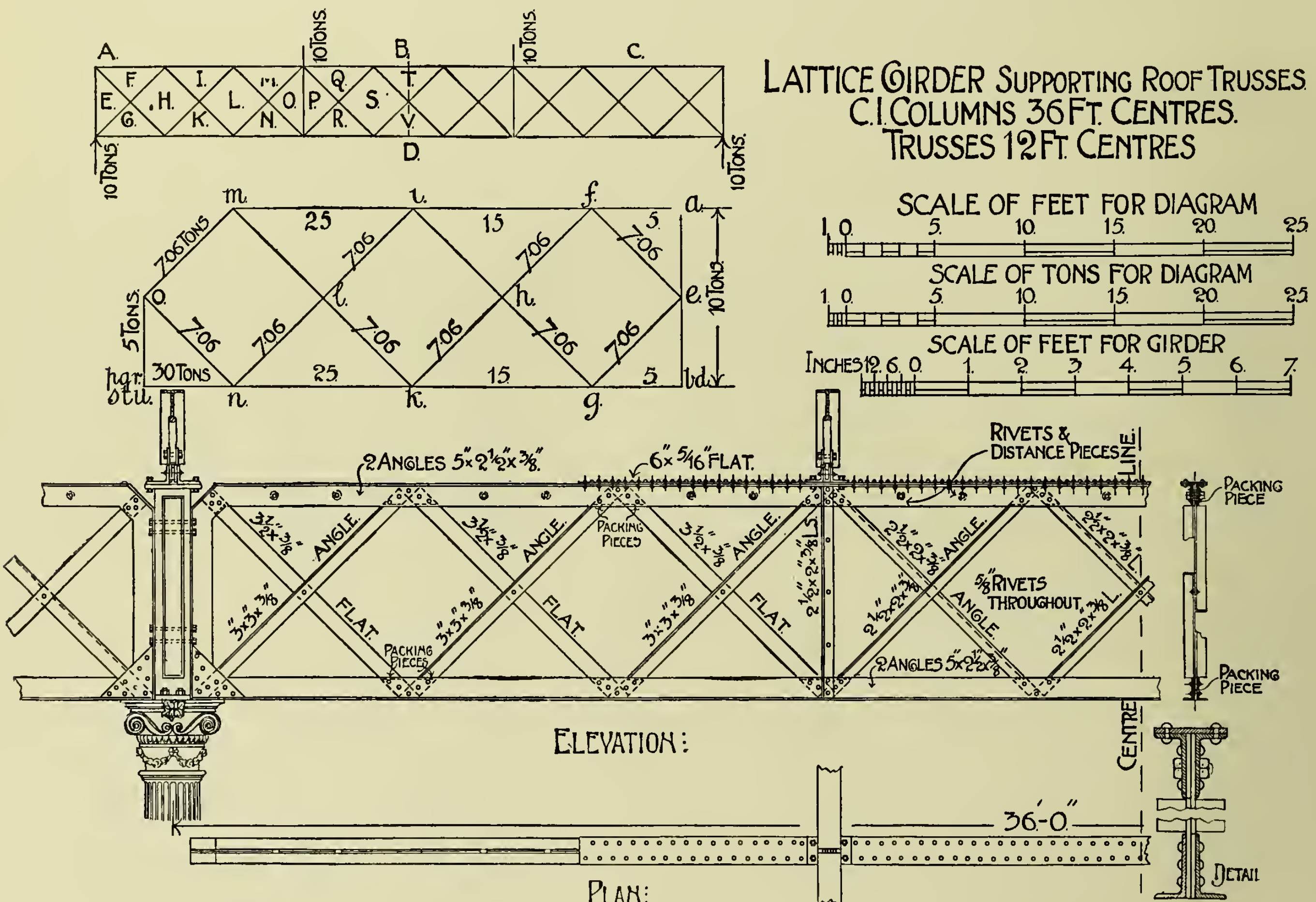


FIG. 127.

those between the loads remain unstressed. This result is identical with that shown in the shear diagram at G, Fig 64, for the sole duty of the diagonal members is to provide for the shear stress.

To resist the tensional stress of 7.06 tons, flat bars, each $3\frac{1}{2} \times \frac{3}{8}$ inches, have been selected, having a safe tensional resistance, after deducting metal for one rivet hole, of 7.01 tons, allowing $6\frac{1}{2}$ tons per square inch.

Before the necessary section of the struts can be arrived at it is first necessary to decide what may be considered as their effective lengths. Each strut and tie is riveted together at the points where they cross. The ties, by tension upon their ends and by their own resistance to bending, add considerably to the rigidity $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ inches would be necessary. Angles $3 \times 3 \times \frac{3}{8}$ will therefore be used, thus halving the difference and ensuring the rigidity of the structure.

As mentioned above, according to the stress diagram the central diagonal members are without stress; but these members cannot be omitted, for the weight of workmen will necessitate their existence, besides which they are important in adding rigidity to the girder. Consequently these members have been shown as composed of angles $2\frac{1}{2} \times 2 \times \frac{3}{8}$ inches, this being approximately the smallest angle that can be used with $\frac{5}{8}$-inch rivets. For the sake of workshop convenience, unless a large number of these girders were required no variation would be made in the size of the diagonal members throughout the whole length of the girder.

The vertical struts may be formed of two angles $2\frac{1}{2} \times 2 \times \frac{3}{8}$ inches, which will be found amply strong enough to resist the stress of 5 tons. Their ends should be forged to fit against the angles of the flanges.

Joints.—The size of the rivets has been assumed to be $\frac{5}{8}$ inch ; but the choice of rivet must go hand in hand with the selection of the members and the design of the joints ; in fact, no part of a design should be considered as definitely settled until the design is complete and its agreement with all other parts of the design has been established.

To resist the shear of 7 tons, three $\frac{5}{8}$-inch rivets will be necessary to connect the diagonals to the flanges, and being in double shear they will have a total resistance of 8.04 tons (see Table at end of Chapter VI.). The bearing stress brought upon the rivets is found to be 10 tons per square inch. In the table just referred to, 9 tons per square inch was considered as safe shearing stress, but 10 tons is very generally adopted ; and it may be considered to be quite safe to use three rivets in the present case. As three rivets are necessary, it is very important to ensure there being room for their insertion in the sizes of members adopted, and it is this consideration which will very largely govern the proportions of members used.

Two rivets will be found necessary to form the joints of the vertical struts, and two rivets have likewise been used at either end of the central diagonal members.

A Design for a Lattice Girder Bridge.—Fig. 130 is a design for a lattice girder bridge to connect two portions of a warehouse which are 60 feet apart. The bridge has been made 7 feet wide between the centres of girders, and has been designed to carry a uniform distributed load of $2\frac{1}{2}$ cwts. per square foot (see p. 135).

The Type of Girder selected is a Warren girder of four systems. This type has the advantage that the multiple web members form a suitable side to the bridge. Any multiple of the simple type may be similarly used.

The Depth of lattice girders is generally relatively greater than that of plate girders, depths as great as $\frac{1}{4}$ of span being sometimes employed. A depth of $\frac{1}{8}$ to $\frac{1}{12}$ of span is, however, mostly generally used.

In the present case the depth of girders has been made $\frac{1}{12}$ of span, and they are thus approximately 5 feet deep, which is a convenient height as forming the sides of the bridge.

Loading.—Allowing for the depth of the bearings at either end of the girders, the effective span may be assumed to be 62 feet. Then total live load $= 62 \times 7 \times 2\frac{1}{2}$ cwts. $= 54$ tons, 5 cwts.

The weight of the bridge will probably be, say, 8 or 9 tons, or for sake of simplicity the total weight of bridge and live load may be taken as 64 tons. Then total load upon one girder $= 32$ tons, and this may be considered as being distributed between every joint in the lower flange, where it will be $\frac{32}{24} = 1\frac{1}{3}$ ton. The loading is shown in the frame diagram at the head of Fig. 130.

Stress Diagram.—Before the stress diagram can be proceeded with it is first necessary to ascertain the stress upon the "End Pillar" B_1C_I, B_1D_1, and to do this the framework must be mentally split up into its component parts, as explained in Chapter X., and as shown in Fig. 128.

At first sight it would appear that component No. 3 should be again split up as in Fig. 129; but the end

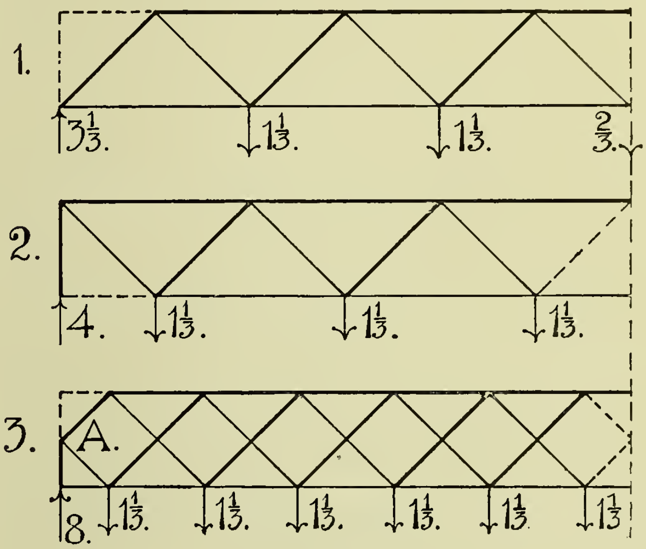


Fig. 128.

bays are here seen to be incompletely framed, and in fact these two portions are interdependent and may be considered as a single system. There is no difficulty in finding the stresses in component No. 3, for the end pillar must obviously carry the whole load brought by this component, which in this case is 8 tons. Thus at joint A only two unknown forces exist, and these

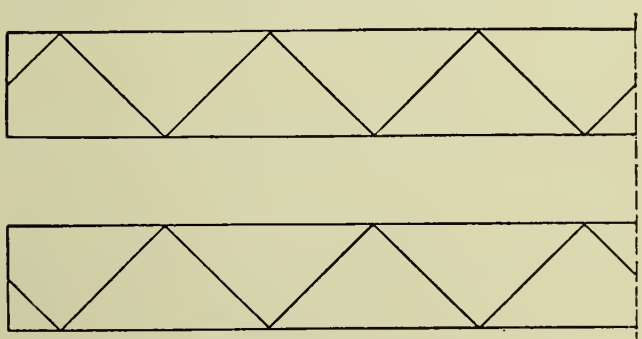


Fig. 129.

may be ascertained by the triangle of forces in the usual way.

Component No. 2 shows that the end pillar is in compression to the extent of 4 tons, while component No. 3 shows that the lower half has an additional load of 8 tons. Thus the upper half of the end pillar has a stress of 4 tons, while the lower half has a stress of $4 + 8 = 12$ tons. Having ascertained this, the stress diagram, as shown in Fig. 130, may be proceeded with as usual.

Flanges.—The maximum stress in tension flange $= 47.3$ tons, which at a safe stress of $6\frac{1}{2}$ tons per square inch requires 7.28 square inches sectional area. This has been supplied by two angles $4 \times 3\frac{1}{2} \times \frac{1}{2}$ inches and one flat $9 \times \frac{5}{16}$ inches, which, after allowing for two $\frac{3}{4}$-inch rivets in the horizontal portions and one $\frac{3}{4}$-inch rivet in the vertical portion, give just 8 square inches. This is apparently slightly more than is necessary; but it may be observed that the vertical 9-inch flat will partake of the nature of the web of a plate girder, and cannot be depended upon to take a uniform stress throughout its depth.

In the compression flange the maximum stress $= 48$ tons, which will be amply met by the section as designed for the tension flange, the distance between the joints being so small in comparison to the size of the member that the full $6\frac{1}{2}$ tons per square inch might be allowed.

Stiffeners.—The compression flange, as stated above, is amply strong enough to resist any tendency to bend between the joints; but it is still possible for it to bend in a horizontal direction, in which direction the web members will assist it only to a small extent. To prevent this tendency to bending, inclined stiffeners are introduced, being riveted to the upper flange at one end and to the angles which carry the floor of the bridge at the other end, certain of these angles being extended for this purpose

The sectional area of the flange (rivet holes not being deducted) $= 9.8$ square inches.

$\therefore$ Stress upon the metal $= \frac{48}{9.8} = 4.9$ tons per square inch.

According to the table at the end of Chapter VIII., 4.9 tons per square inch is the safe load when $\frac{l}{r} = 35$.

According to the table on page 89; for T-section $r = .21\ \mathrm{D} = .21 \times 8\frac{1}{2} = 1.8$ inch.

$\therefore$ 4.5 tons is a safe load if $l = 35 \times 1.8 = 63$ inches.

According to this, the stiffeners should be placed 63 inches apart, or at every other joint in the flange; but the safe loads given for the lower values of $\frac{l}{r}$ in the table probably err rather on the side of safety, besides which the web members will add slightly to the stiffness of the flange. On these considerations it will be safe to place the stiffeners at every third joint, or 7 feet 8 inches apart. Towards the ends of the girders they might be spaced farther apart; but for simplicity they have here been spaced equally.

Web Members.—The maximum stress upon the braces $= 5.7$ tons. This has been met by flat bars $3 \times \frac{5}{8}$ inches, each of which, allowing for one $\frac{3}{4}$-inch rivet hole in its cross section, has a safe strength of 5.88 tons.

The effective length of the struts has here been considered to be half their total length, or 36 inches (see p. 108). The smallest size of angle in which $\frac{3}{4}$-inch rivets can be conveniently used is 3 inches, and a section $3 \times 2 \times \frac{3}{8}$ inches may be selected. Approximate value of $r = .2\mathrm{D}$.

$\therefore r = .2 \times 2 = .4$ inch; and $\frac{l}{r} = \frac{36}{.4} = 90$.

Then safe stress per square inch $= 2.8$ tons. Sectional area of strut $= 1.73$.

$\therefore$ Safe load $= 4.85$ tons.

4.7 tons is the greatest load upon the struts, except upon strut E_1H_1, C_1F_1, which, being considerably shorter than the other struts, will be amply strong enough formed of the same section.

According to the table at the end of Chapter VI., $\frac{3}{4}$-inch rivets have a single shear strength of 2.21 tons, and a resistance to bearing in $\frac{5}{16}$-inch plates of 2.11 tons. Thus to resist the force of 5.7 tons, three $\frac{3}{4}$-inch rivets are necessary, as shown upon the drawing. The number may be reduced to two when the stress in the web members is not more that $2 \times 2.11 = 4.22$ tons.

With the evenly distributed load assumed, all members sloping away from the centre of the girder down towards the abutments are in compression, while those crossing them are in tension. However, were the load distributed unevenly, some of the members that have been found to be in tension would be put into compression. Towards the centre of the girder the struts have been reduced to $2\frac{1}{2} \times 2 \times \frac{3}{8}$ inches, and for the reason just given, as well as for the sake of producing as few variations in the size of members as possible, the same section has also been used for the braces over this part of the girder.

The employment of $2\frac{1}{2} \times 2$-inch angles necessitates the use of $\frac{5}{8}$-inch rivets. These have single shear resistance of 1.53 ton, and bearing resistance in $\frac{5}{16}$-inch plate of 1.76 ton. Thus joints may be made with two $\frac{5}{8}$-inch rivets when the stress in the web members is not more than $2 \times 1.53 = 3.06$, and it is at this point that the reduction in the size of the struts has been made.

It will be noticed that two sizes of rivets have here been used in order to introduce lighter web members; however, the practice of varying the size of rivets should be avoided as far as is consistent with economy.

End Pillars.—The end pillar as shown is, of course, out of all proportion to the load it has to carry, but it is desirable to have a rigid termination to the girder.

Bearings.—The girder rests upon plates 14×12 inches, which, with the load of 16 tons brought by the girder, produces a load of 13.7 tons per square foot upon the stone (see Table, end of Chapter IV.). The bed stone has been made 1 foot $10\frac{1}{2}$ inches $\times$ 1 foot $10\frac{1}{2}$ inches to suit brick dimensions, and produces a load upon the brickwork of 4.5 tons per square foot.

Cross Beams.—The live load on each cross beam $=$ 54 tons, 5 cwts. $\div 24 = 45$ cwts. The total load on each cross beam including the weight of flooring will be 46 cwts. The cross beams may be formed of angle, tee, channel or joist, according to the load to be carried. Angle and channel sections have the advan-

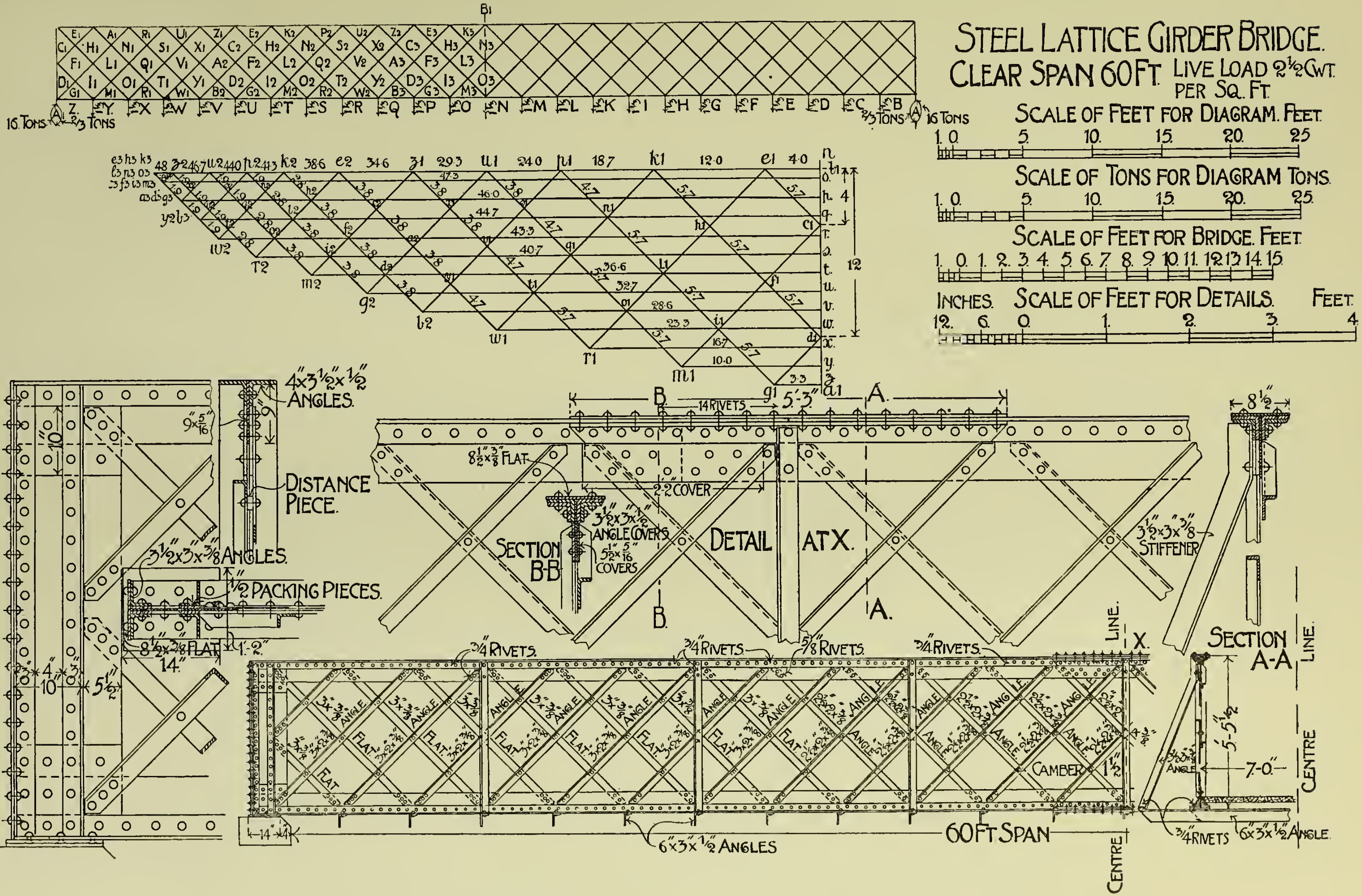

STEEL LATTICE GIRDER BRIDGE.
CLEAR SPAN 60 FT.
LIVE LOAD 2½ CWT. PER SQ. FT.
SCALE OF FEET FOR DIAGRAM. FEET.
SCALE OF TONS FOR DIAGRAM TONS.
SCALE OF FEET FOR BRIDGE. FEET.
INCHES. SCALE OF FEET FOR DETAILS. FEET.
16 TONS
2⅔ TONS
4"x3½"x½" ANGLES.
DISTANCE PIECE.
3½"x3"x⅜" ANGLES.
½" PACKING PIECES.
8½"x⅜" FLAT
14"
1'-2"
8½"x⅜" FLAT
2'2" COVER
14 RIVETS
5'-3"
SECTION B-B
3½"x3"x½" ANGLE COVERS
5½"x5/16" COVERS
DETAIL AT X.
3½"x3"x⅜" STIFFENER
SECTION A-A
¾" RIVETS.
⅝" RIVETS.
LINE.
CENTRE
CAMBER 1½"
60 FT SPAN
6"x3"x½" ANGLES
¾" RIVETS
6"x3"x½" ANGLE.
7-0"
5'-5½"

FIG. 130.

tage of greater width in the flanges through which to rivet the connections.

Selecting an angle $6 \times 3 \times \frac{1}{2}$ inches, the moment of inertia $= I = 15.7$, while $y = 3.81$.

$\frac{Wl}{8} = I\frac{f}{y}$; and taking f. at $7\frac{1}{2}$ tons (see p. 80),

$$\frac{W \times 7 \times 12}{8} = \frac{15.7 \times 7\frac{1}{2} \times 20}{3.81}.$$

$$\therefore W = 59 \text{ cwts.}$$

On account of the unsymmetrical nature of an angle about its vertical axis, the safe load will be approximately only $\frac{3}{4}W = \frac{3}{4} \times 59 = 44$ cwts.

The section chosen then is evidently suitable.

Each cross beam is attached to the two girders by two $\frac{3}{4}$-inch rivets in each. This will be ample; but too much reliance should not be placed upon the resistance of rivet heads, for the initial stress set up in the rivet on cooling is an uncertain quantity.

Flange Joints.—The length of the flanges is obviously too great to be constructed in a single length. The joint in the compression flange is shown in detail in Fig. 130, while that in the tension flange is similar to it. The angles composing the flange, having a sectional area of 7 square inches, have been covered with two angles $3\frac{1}{2} \times 3 \times \frac{1}{2}$ inches and a flat $8\frac{1}{2} \times \frac{3}{8}$ inches, which have a total sectional area of 9.2 square inches.

All the rivets are in double shear, giving a safe shear resistance of 3.87 tons per rivet (Table, end of Chapter VI.). The horizontal arms of the angles present a thickness of $\frac{1}{2}$ inch to resist bearing, giving a safe resistance of 3.37 tons. In the vertical arms the thickness $= 1$ inch, which gives a resistance to bearing of 6.75 tons per rivet. The least of these amounts must be used in the calculations.

The stress that the angles of the flange are capable of carrying $= 7 \times 6\frac{1}{2} = 45$ tons.

The number of $\frac{3}{4}$-inch rivets necessary on either side of the joint to transmit this stress $= \frac{45}{3.37} = 14$, four of these being placed in the vertical arms of the angles and ten in the horizontal arms.

That part of the $\frac{5}{16}$-inch web which comes below the angles has been covered with double covers $\frac{5}{16}$ inch thick. The amount of metal supplied by this arrangement is unnecessary; but, as has been mentioned before, it is advisable to regard $\frac{5}{16}$ inch as the minimum thickness for metal in structural steelwork, besides which $\frac{5}{16}$-inch metal is already required for the work and should be used on this account. $\frac{5}{8}$-inch rivets have been used for the web members in this part of the girder, and may here be employed to form the joint. The safe bearing resistance of $\frac{5}{8}$-inch rivets in $\frac{5}{16}$-inch plate $= 1.76$ ton. Area of web $= 2.8$ square inches, having safe strength $2.8 \times 6.5 = 18$ tons. Number of rivets required on either side of joint $= \frac{18}{1.76} = 10$. Six of these have been placed in the covers, while the remaining four $\frac{5}{8}$-inch rivets have been replaced by three $\frac{3}{4}$-inch rivets through the angle covers.

The angle covers have thus to be long enough to take 7 rivets in the vertical arm to the left of the joint, and 4 rivets to the right of the joint; but, for sake of uniformity, it has been made of equal length on either side of the joint.

A somewhat neater joint might have been provided by making the angles "break joint"; but the joint as arranged has the advantage of simplicity.

Camber.—It is the custom to slightly camber lattice girders in order to avoid the possibility or the appearance of sagging.

A camber of 1 inch in every 40 feet of span is a very usual allowance. The present example, having a span of 60 feet, has been given a camber of $1\frac{1}{2}$ inch.

In constructing the girder the camber may be obtained by slightly increasing the length of every strut, and correspondingly decreasing the length of the ties. Another method is to slightly increase the length of the upper flange, the lengths of the struts and ties remaining unaltered. The amount of variation in the lengths of the flanges rendered necessary by the latter method may be found as follows.

Radius of curvature of flanges $= R = \frac{\left(\frac{s}{2}\right)^2 + c^2}{2c}$; where $s =$ span and $c =$ amount of camber (both in same units). Then, neglecting c^2 as inappreciable,

$$r = \frac{s^2}{8c}.$$

Thus in the present case $r = \frac{(60)^2}{8 \times \frac{1}{8}} = 3600$ feet, while radius of upper flange $= 3600 + 5 = 3605$ feet. Then length of top flange : length of bottom flange :: outer radius : inner radius.

$$\therefore \text{If } l = \text{length of top flange, } \frac{l}{60} = \frac{3605}{3600}.$$

Whence $l = 60$ feet 1 inch.

The extra length of 1 inch has to be divided among the portions of flange between every joint.

Plate girders may also be given a camber in cases where the web is to be made up in several lengths, but in ordinary cases it is quite unnecessary.

CHAPTER XII

ROOFS

A Roof Truss is a girder which supports a roof at a suitable angle, and, as in the case of the trusses previously considered, complete triangulation is necessary.

Several forms of roof truss are shown in Fig. 133, together with spans suitable for each.

The principal considerations which affect the choice of the particular form of truss to be used are the width

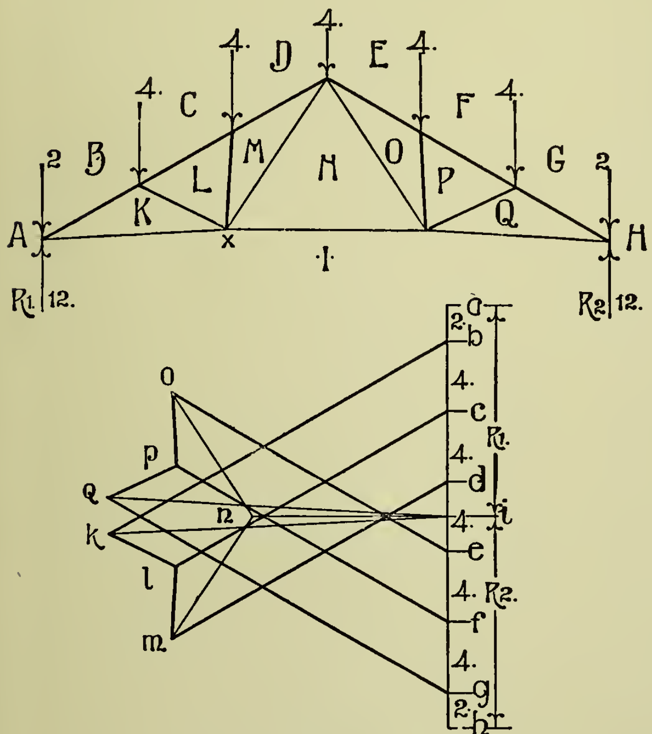


Fig. 131.

of span and the greatest length of unsupported rafter which is advisable, the latter being generally from 5 to 12 feet. The struts should be as short as possible, and the truss should be light. Appearance and the fancy of the designer also govern the selection.

The *force diagram* of a truss with assumed loading is shown in Fig. 131. The method of procedure is the same as was described in Chapter X. The loads are set down at *ah*. Starting at the left-hand side of the truss, the parallelogram *iabki* is drawn with its sides parallel to the reaction R_1, load 2, and members BK and KI. Then the parallelograms *kbclk* and *lcdml* are drawn; after which the parallelogram *iklmni* is obtained, as there are now only two members at joint at *x* which are still unknown.

"Principle of Moments," or "Method of Sections."—Fig. 132 shows a case similar to the last. Imagine the truss to be cut in two by a line such as XX. Then each half will remain in equilibrium if forces be applied to the ends of the cut members equal in magnitude and direction to the stresses in those members. If any number of forces are in equilibrium the algebraic sum of their moments about any point is equal to o. The section should be made, if possible, through three members. Then to find the stress in one member, take moments about the point of intersection of the other two. Thus section XX cuts through members 2, 8, and 7. 2 and 7 intersect at A, and, taking moments about

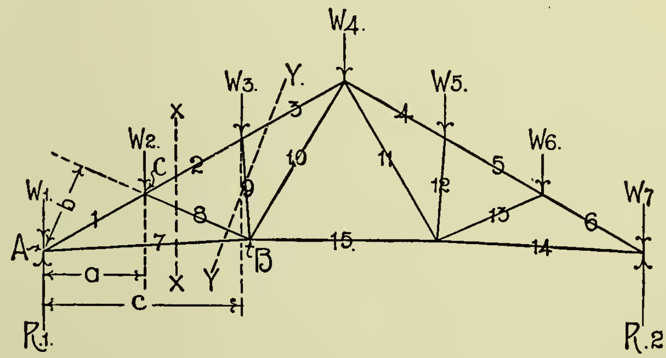


Fig. 132.

this point and writing S_8 for the stress in member 8, $W_2a - S_8b = 0$, or $S_8 = \frac{W_2a}{b}$ (see Chapter I.). Similarly the stresses in members 2 and 7 may be found by taking moments about B and C respectively. The stresses in other members can be found by making other sections, and by a similar procedure. To find the stress in the member 9 it is necessary to make a section through four members as at YY; however, the stress in one of these, S_8, has already been found, while the moments may be taken about A, the point of intersection of two other members. Then

$$W_2a + W_3c - S_8b - S_9d = 0.$$

$$\text{Or } S_9 = \frac{W_2a + W_3c - S_8b}{d}.$$

This method of finding the stresses in the members of a truss is chiefly useful when the stresses in one or two members only are required.

It may be noted that in Fig. 132 the length *d* has

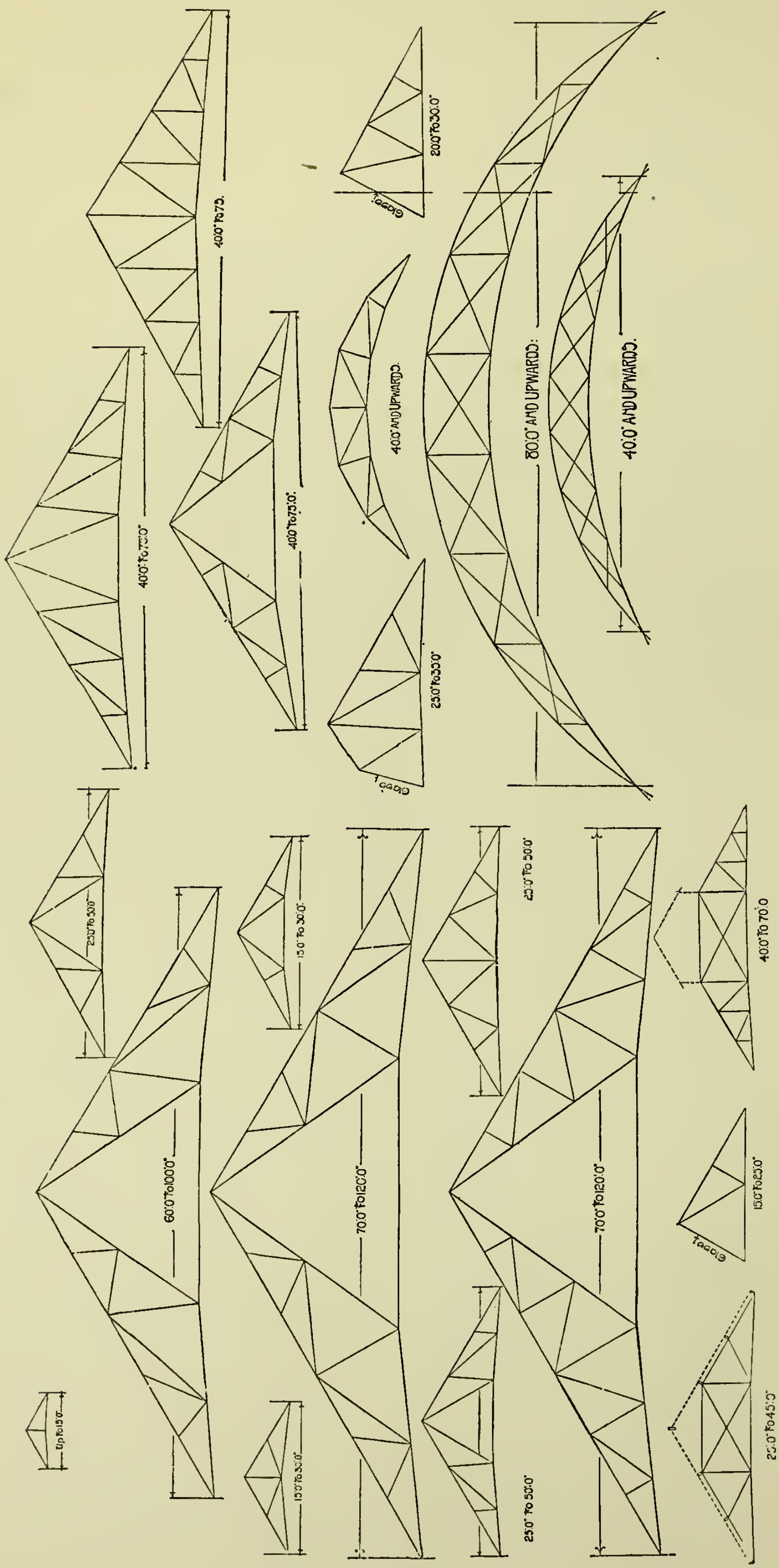


Fig. 133.

not been indicated, to avoid confusion on the diagram, but it is obtained by dropping a perpendicular from A to the line 9.

Fig. 134 shows what is known as a "Fink" truss, with its force diagram. Starting at the left extremity of the truss, the force diagram is proceeded with in the ordinary way until the fourth or fifth joint is met with, when three unknown members are found. The diagram may be continued by a system of trial and error, and when its symmetrical nature is once known further cases will present no difficulty. It will, however, be more satisfactory at the outset to find the stress in one member by the method of moments described above. Thus making section XX, and taking moments about the apex of the truss, S representing the stress in SL,

$$W_1d_1 + W_2d_2 + W_3d_3 + W_4d_4 + Sh - R_1d_1 = 0.$$

Whence

$$S = \frac{R_1d_1 - W_1d_1 - W_2d_2 - W_3d_3 - W_4d_4}{h}.$$

Having found the stress in this member, the diagram may be proceeded with as usual.

Wind Loads.—In the cases taken above, the vertical loading only has been dealt with, but the effect of wind load upon the stress diagram will now be considered, presuming that it acts on one side of roof only, and normally to the surface.

In Fig. 135 the loading is supposed to be entirely due to wind upon one side of the roof. In the cases previously considered, the loading being vertical and evenly distributed, the reactions were equal. The reactions in this case are unequal, and may be found by the method described at the end of Chapter I. As the line of direction of each load is the same, the reactions must be parallel to them. The funicular diagram, as shown in a dotted line, may then be started at any point in the line of direction of either reaction, the polygon being completed by joining XY. Drawing o_1f parallel to XY upon the polar diagram, the magnitude of the reactions is found. The reactions

found, the force diagram may be drawn in the usual way, as shown. It should be noted that, space A being bounded by forces which are in one and the same straight line, the line o_1a does not appear.

In Fig. 136 the load on the left side is presumed to be entirely due to the wind, while the load on the opposite side is due only to the weight of the roof. By setting down these loads on the force diagram, and by completing the parallelogram, the total reaction of the two abutments is found to be equal in amount and direction to *ha*. Having found the direction of the reactions, these may be drawn on the frame diagram. The total reaction may then be divided up into its two parts by means of the polar and funicular diagrams shown dotted, the funicular diagram being started at any point in the line of direction of either reaction. The stress diagram is continued as usual.

Roller Bearings.—In the two cases last considered the ends of the trusses were assumed to be fixed, but in roofs of large span, say 80 feet or more, one end of the truss should be supported on rollers, to allow of expansion due to temperature and to prevent the truss from exerting a thrust upon the walls. The reaction of

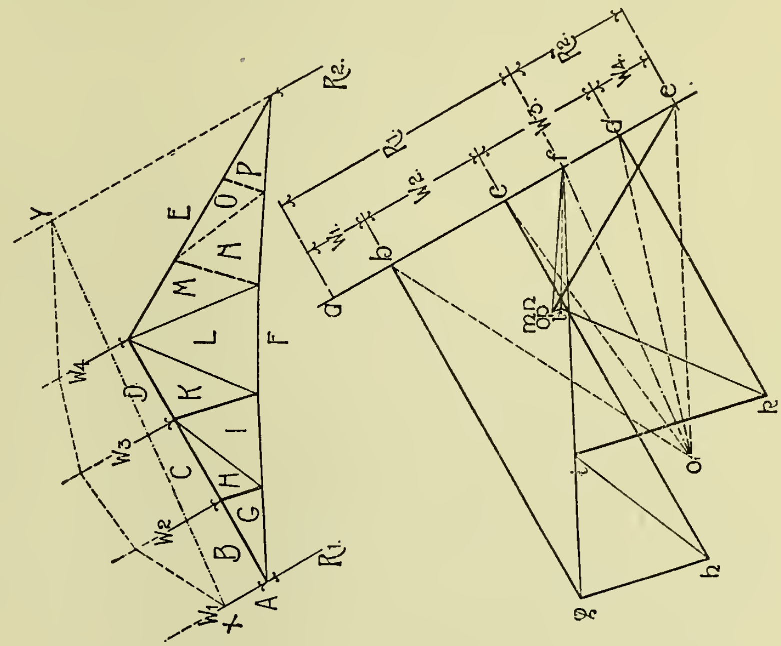


Fig. 135.

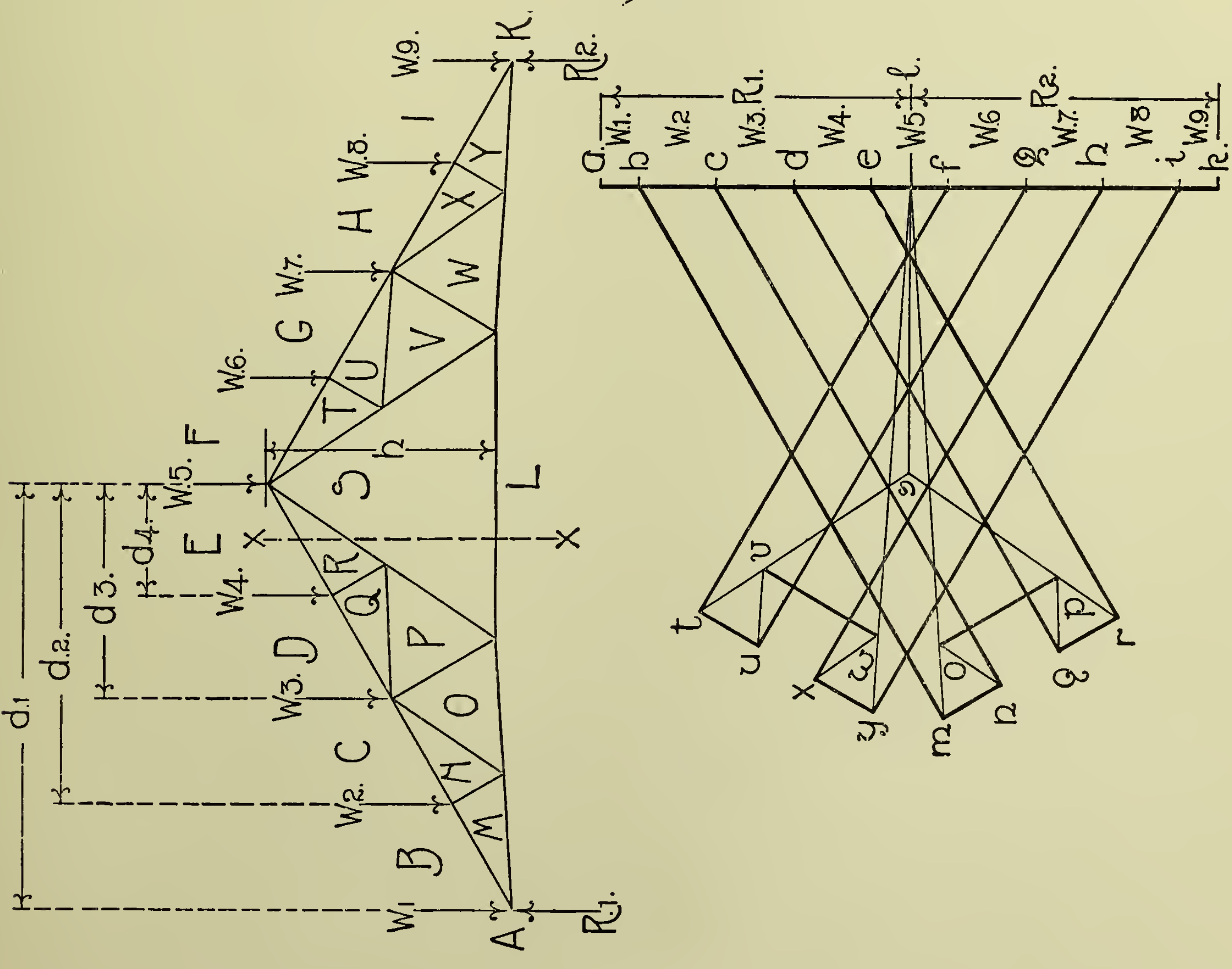


Fig. 134.

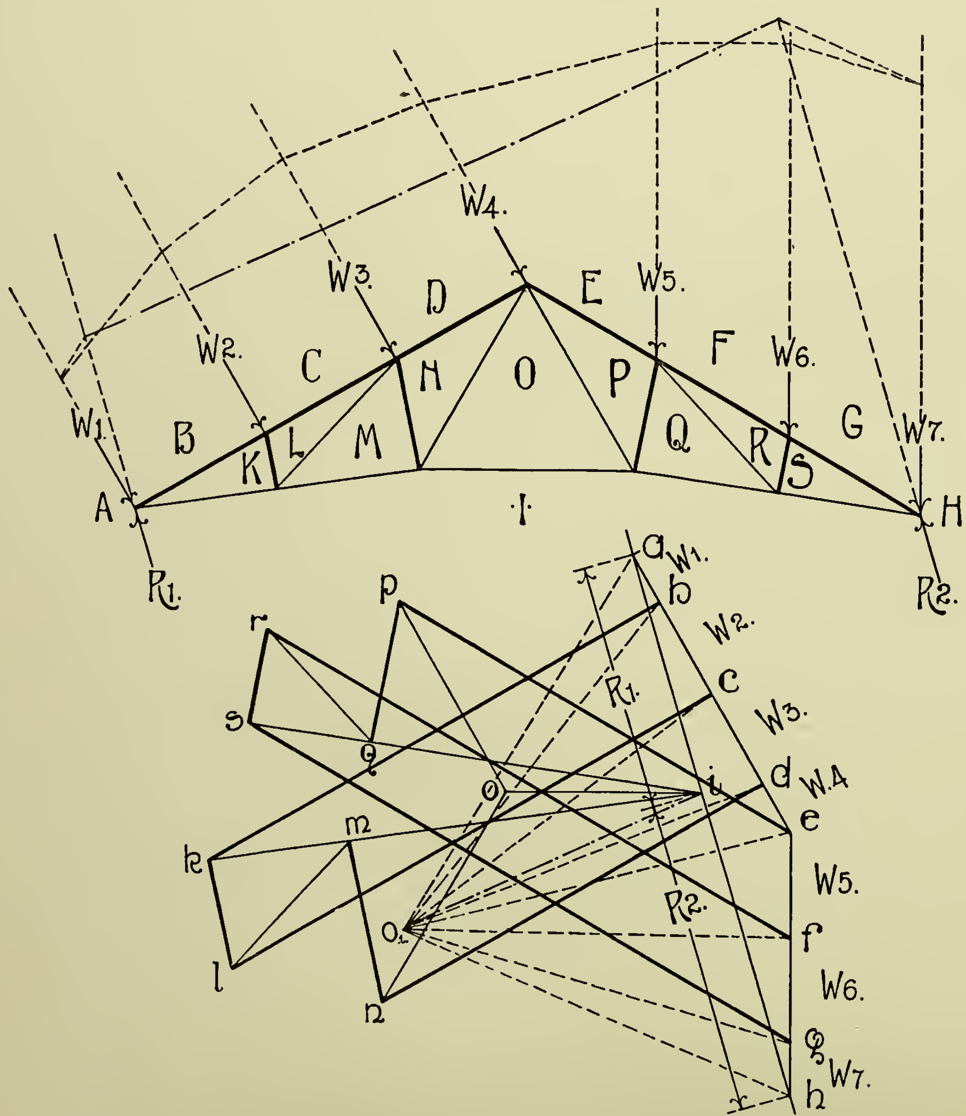


Fig. 136.

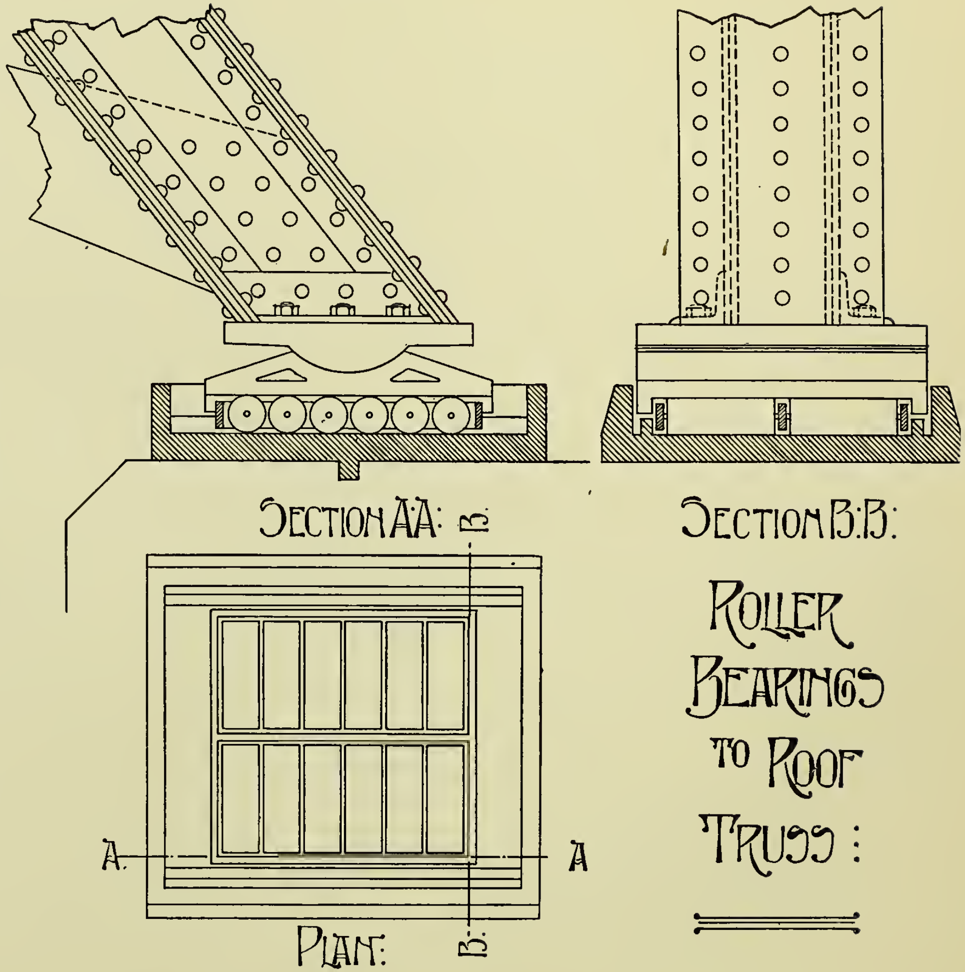


Fig. 137.

the abutment under the rollers can obviously only be vertical.

Fig. 137 shows the details of a roller bearing for a large arched roof.

In Fig. 138 a case similar to that in Fig. 136 is given, one end being here assumed to be "free," or supported polygon of loads and reactions *aehia*. The stress diagram may now be continued as usual.

Fig. 139 shows the same truss with wind load coming from the opposite side. The funicular polygon must again be started from point X, and R_1 and R_2 are now found to be as shown on the force diagram.

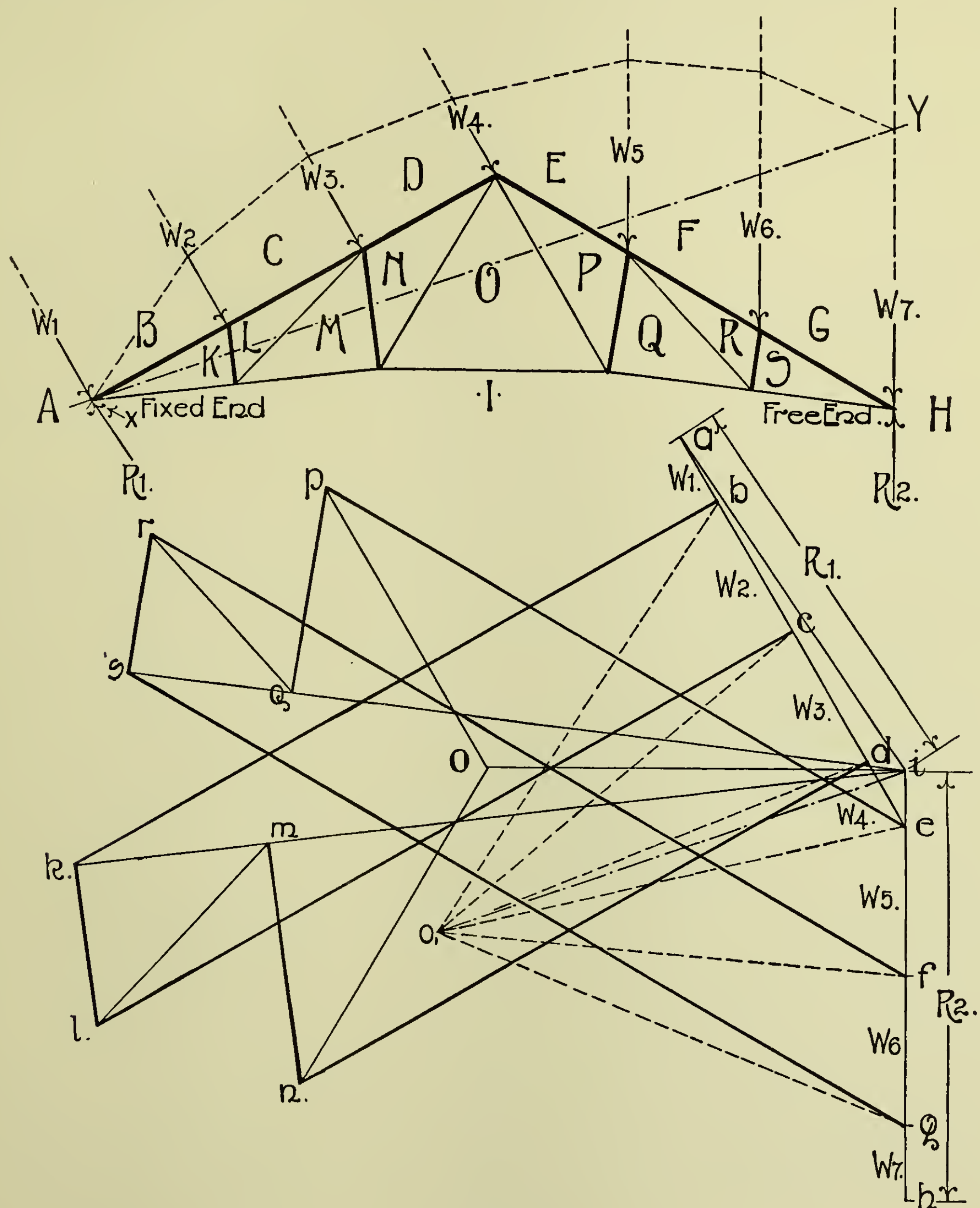


Fig. 138.

on rollers. The reaction at the right-hand end is thus known to be vertical, as shown. In order to draw the funicular diagram it must be started at point X, for this is the only point in R_1 which is definitely known. The polygon is then continued as usual, and by joining the extremities XY, and by drawing o_1i parallel to it on the force diagram, R_2 is cut off on the vertical line *hi*, while R_1 is found by joining *ai*; thus completing the

The stresses in some of the members are seen to vary considerably in the last two examples, notably that in the member OI.

Roof trusses of less than 80 feet span should likewise be fixed at one end only, the shoe at the other end resting upon an iron plate and fastened down by bolts passing through slotted holes in the shoe, so that motion is still possible (see Fig. 148). Although one

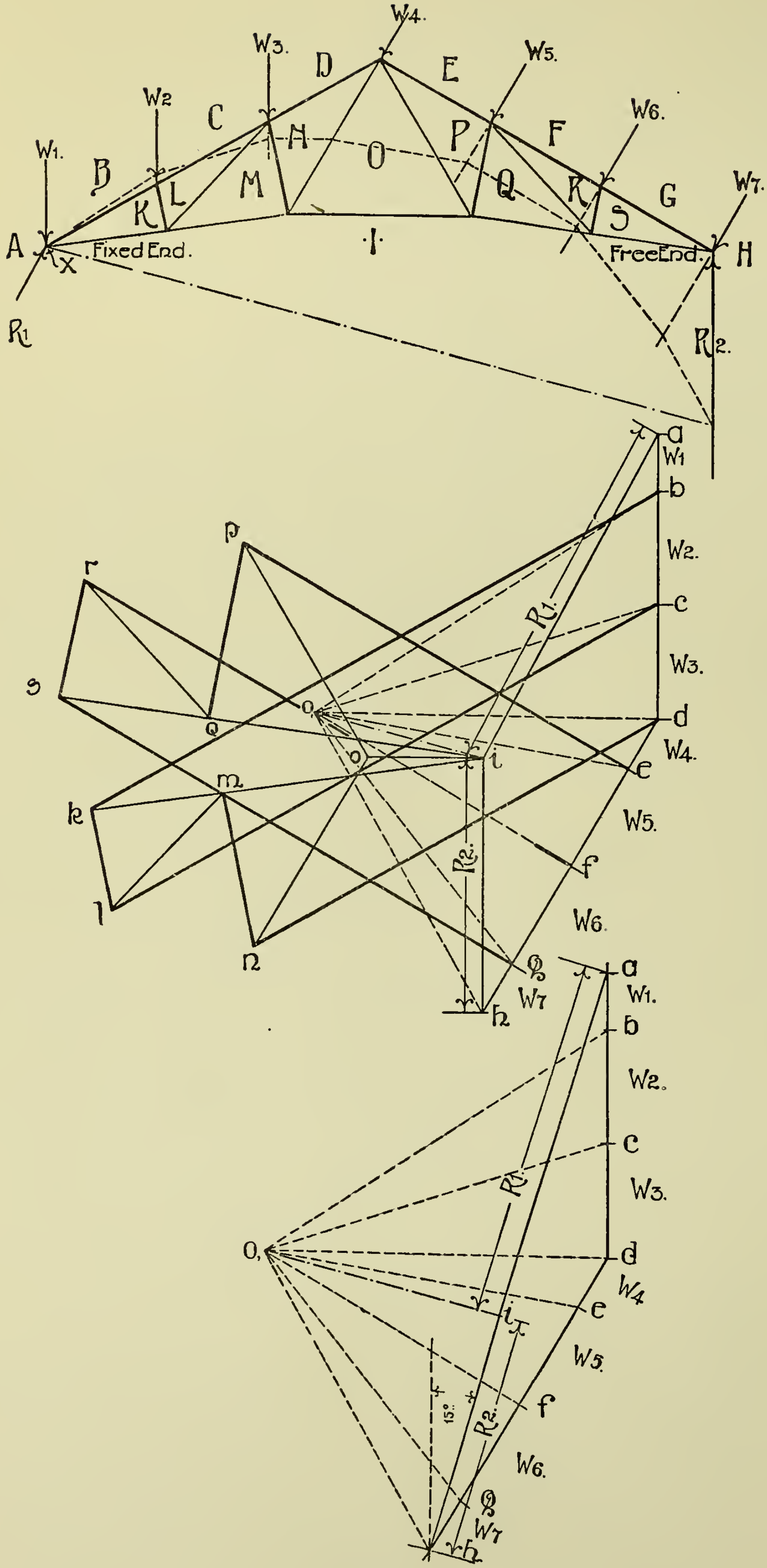


Fig. 139.

end is not fixed, the friction between the plates is sufficient to cause the reaction at this end to act towards the resultant pressure. The coefficient of friction between smooth steel surfaces is approximately equal to tan 10 degrees, but the surfaces will probably be rusty or rough, and the coefficient will then be much increased. As the resultant pressure will seldom differ more than 30 degrees from the vertical, no great error will be made if both ends be considered to be fixed. Otherwise the resultant at the free end may be drawn at say 15 degrees from the vertical, as is shown in the lower funicular and polar diagrams attached to Fig. 139.

The Nature of Loading on Roof Trusses. — Before the stresses in the members of a roof truss can be found the load brought to bear upon the truss must first be known.

A truss has to carry—1st, a dead load composed of the weight of the actual roof as well as the weight of the truss itself; 2nd, a live load produced by wind, snow, or workmen effecting repairs.

The portion of a roof carried by a single truss is shown hatched in isometric projection in Fig. 140, while the portions carried at each joint are shown hatched in opposite directions. The dead load due to the roof itself may be found when the materials composing the roof are known as well as the weights of those materials. A short list is here given:—

Slates, Countess .	8 lbs.	per	sq. ft.
Tiles, plain, 3½-inch lap, soaked with rain .	18	,,	,,
Tiles, pan, 3-inch lap, pointed in mortar and soaked with rain . . .	12	,,	,,
Slate battens for Countess slating . .	1	,,	,,
Boarding, ¾ inch .	2½	,,	,,
Boarding, 1 inch .	3½	,,	,,
Common rafters, 4 × 2 × 12-inch centres .	2½	,,	,,
Corrugated iron, 16 W.G. . . .	3½	,,	,,
Corrugated iron, 18 W.G. . . .	2½	,,	,,
Corrugated iron, 20 W.G. . . .	2	,,	,,
Corrugated iron, 22 W.G. . . .	1¾	,,	,,

The weight of the truss itself cannot be found accurately until it has been designed, so that for sake of calculation its weight must be assumed.

For trusses between 20 and 80 feet span, and spaced 8 to 12 feet apart, the weight per square foot of surface may vary from $1\frac{1}{2}$ to $6\frac{1}{2}$ lbs.; while the total weight, including corrugated iron covering, may be assumed to be from 6 to 12 lbs. per square foot, or from 16 to 22 lbs. when covered with slates.

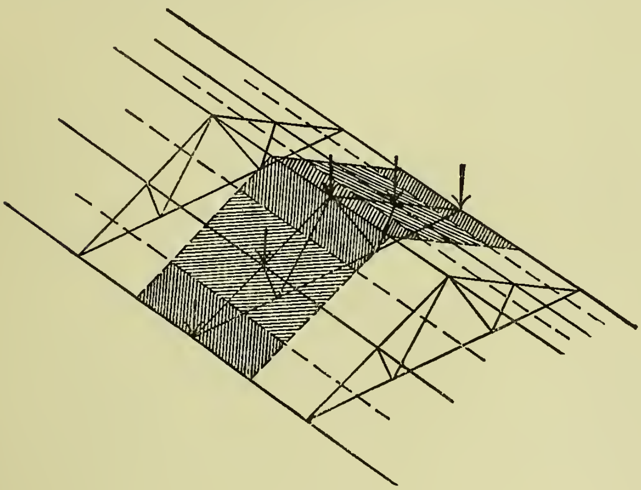


FIG. 140.

The effect produced by wind is a very uncertain quantity. It has been laid down by the Board of Trade that 56 lbs. wind pressure per vertical square foot shall be allowed for in designing bridges, but for the ordinary cases of roofs 50 lbs. per square foot may be considered ample in this country. Having decided this point, its effect upon a sloping roof is still uncertain. We know that true fluid pressure acts normally to all surfaces, and it might be thought that, the wind blowing horizontally, the pressure on the roof might be found by breaking up the force into its component parts, parallel and at right angles to the slope of the roof, as in Fig. 141. But the direction of the wind is not always horizontal, and the effect of friction cannot entirely be left out of consideration. The only experiments bearing upon the subject were made about 100 years ago by Dr. Hutton, and gave results differing largely from the theoretic values. The following formula, given by Professor Unwin, is based upon the above-mentioned experiments:—

$$Pn = P \sin i^{1.84 \cos i - 1}.$$

where Pn = normal pressure of wind upon roof.
P = horizon pressure of wind.
i = angle of inclination of roof.

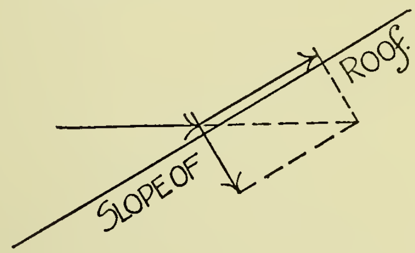


FIG. 141.

The following table is calculated from the above formula, when $P = 50$ lbs. per square foot.

Angle of slope of roof . .	5°	10°	20°	30°	40°	50°	60°
Normal pressure . . .	6	12	$22\frac{1}{2}$	33	$41\frac{1}{2}$	$47\frac{1}{2}$	50

It is usual to consider the wind as acting upon one slope of the roof only; however, there is often a lifting tendency on the side remote from the wind caused by suction. The values in the above table are possibly far from accurate, but they probably err on the side of safety.

The weight of snow or workmen need seldom be considered, as these will not be upon the roof during a gale. A load of 5 lbs. per square foot may be allowed for snow upon roofs of ordinary slope.

CHAPTER XIII

THE DESIGN OF ROOF TRUSSES

Suppose that it is necessary to design a roof truss of 55 feet span to be covered with slates laid upon boarding.

Selecting the type shown in Fig. 134, it may be assumed that the length of the building to be roofed may be divided into 10-foot bays; that is to say, the trusses will be spaced with 10 feet between their centres. It is usual to camber the tie rod to a certain extent, as this improves the appearance of the truss, increases the head room, and shortens the length of the struts; but at the same time it adds considerably to the stresses in the tie and rafter. In the present case a total camber of 2 feet has been given to the tie rod (Fig. 142).

Loading.—The length of rafter = 32 feet approximately.

∴ The total roof surface carried by one truss = $32 \times 10 \times 2 = 640$ square feet. Assuming the truss to weigh 2500 lbs., this will approximately be equal to a load of 4 lbs. per square foot. The dead load may thus be assumed to be composed as follows:—

Truss	4	lbs. per square foot.
Slates	8	,, ,,
$\frac{3}{4}$-inch Boarding .	$2\frac{1}{2}$	,, ,,
Common rafters .	$2\frac{1}{2}$	,, ,,
Purlins . . .	$1\frac{1}{2}$	,, ,,
Say	19	,, ,,

As shown in Fig. 142, this will give a dead load of $13\frac{1}{2}$ cwts. at each joint except at the ends, where there will be half this load.

Assuming the pitch to be 30 degrees, according to table given at end of last chapter, the normal wind pressure will be 33 lbs. per square foot. This will give a load of $23\frac{1}{2}$ cwts. at the three central joints on one side, and $11\frac{3}{4}$ cwts. at the two ends, as shown in Fig. 142. The resultants of the dead and live loads are then found by the parallelogram of forces.

On setting down the loads, the angle that the resultant *ak* makes with the vertical is very small, being less than 15 degrees (*see* p. 118). The reactions may, therefore, be considered to be parallel, and their amounts are found by the polar and funicular diagrams shown dotted.

Stress Diagram.—This, as also the polar and funicular diagrams, should be carefully drawn to a large scale with a fine pointed pencil. On drawing the last line of the diagram the refusal of the polygon to close accurately would prove inaccuracy of draughtsmanship. Much will depend upon the accuracy with which the reactions are found.

Having drawn the stress diagram, the various members should be tabulated somewhat as follows:—

Member.	Tension Cwts.	Compression Cwts.	Necessary Sectional Area.	Form of Member.
ML and YL .	225		1.88	$1\frac{9}{16}''$ Rod.
OL and WL .	181		1.51	$1\frac{3}{8}''$,,
RS and TS .	146		1.22	$1\frac{1}{4}''$,,
PS and VS .	102		.85	$1\frac{1}{16}''$,,
NO and XW .	44		.37	$\frac{11}{16}''$,,
PQ and VU .	44		.37	$\frac{11}{16}''$,,
SL . . .	84		.70	$\frac{15}{16}''$,,
BM and IY .		234	Length	
CN and HX .		$227\frac{1}{2}$	$8'\ 0''$	$4\frac{1}{2}'' \times 4\frac{1}{2}'' \times \frac{1}{2}''$ Tee.
DQ and GU .		$220\frac{1}{2}$		
ER and FT .		214		
OP and WV .		$70\frac{1}{2}$	$7'\ 0''$	$2\text{–}3'' \times \frac{5}{16}''$ Flats.
MN and YX .		$35\frac{1}{2}$	$3'\ 6''$	$2\text{–}2'' \times \frac{5}{16}''$ Flats.
QR and UT .		$35\frac{1}{2}$		

Size of Members.—It is obvious that, as the wind may blow upon either side of roof, both sides of truss must be made similar.

The forked ends of bars shown in the illustration are forged separately, and are then welded to the bars. These welds may weaken the bar by as much as 25 per cent., and a moderately low safe stress must consequently be allowed for. The diameters of rods given in the above table are calculated with a safe tensional load of 6 tons per square inch.

There is now a strong tendency to avoid the use of welds altogether, using riveted flats and angles in the place of rods with forged eyes.

Rafters are generally formed with T-iron or two angles back to back, and sometimes with channel iron or rolled joist. They are nearly always made of the same section throughout. Each portion of rafter lying between joints may be considered as a pillar with hinged ends, except the end portions, which may be considered as having one end fixed; for upon the load coming upon it the rafter will have a tendency to bend as indicated in Fig. 143, which may be compared with Figs. 97 and 99.

A T-section $4\frac{1}{2} \times 4\frac{1}{2} \times \frac{1}{2}$ inches will be selected, and its strength may be calculated as follows. According to table, Chapter VIII., approximate value of $r = .21D = .21 \times 4.5 = .945$ inch.

$\therefore \frac{l}{r} = 100$ approximately, and according to table at

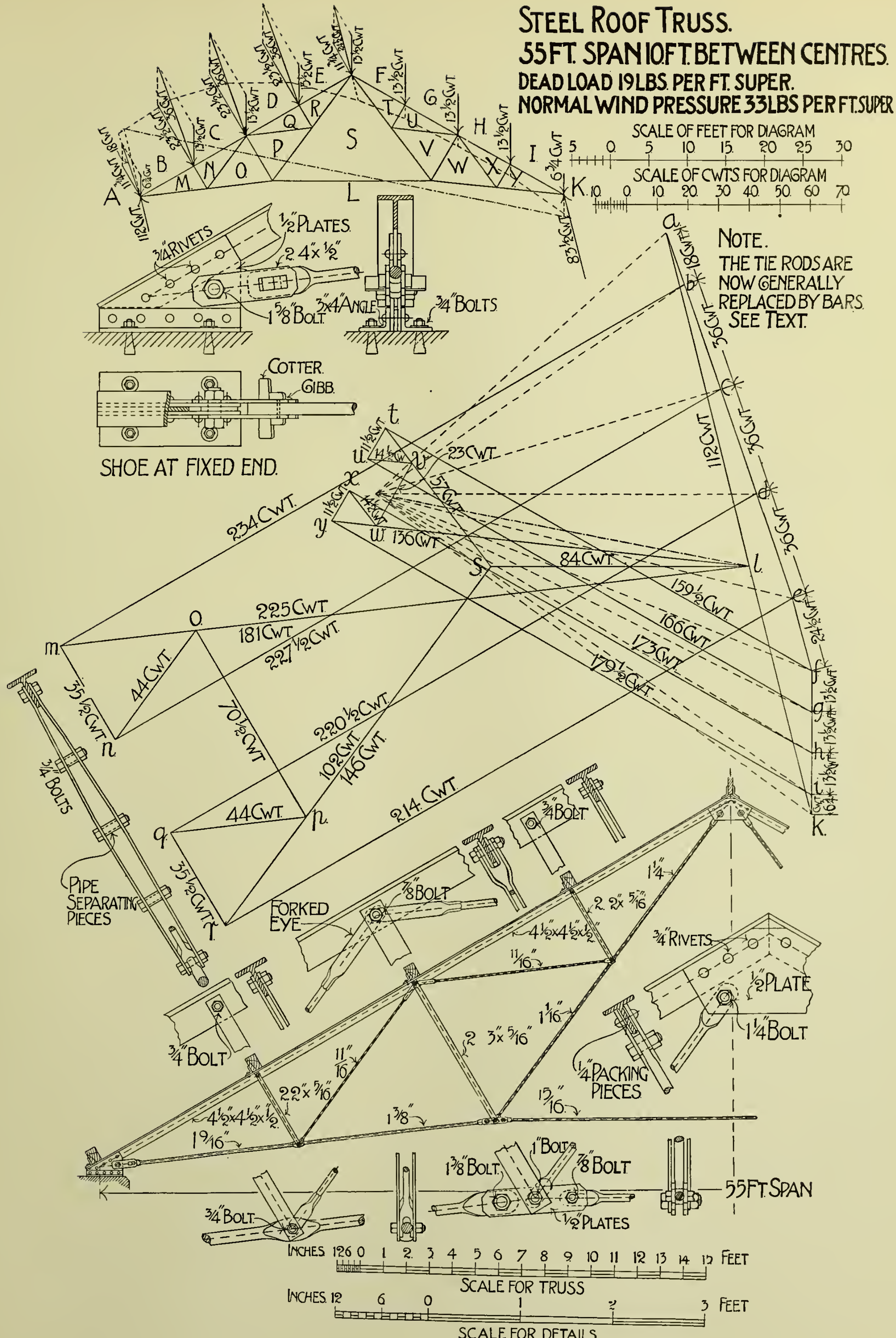

STEEL ROOF TRUSS.
55 FT. SPAN 10 FT. BETWEEN CENTRES.
DEAD LOAD 19 LBS. PER FT. SUPER.
NORMAL WIND PRESSURE 33 LBS PER FT. SUPER
SCALE OF FEET FOR DIAGRAM
SCALE OF CWTS FOR DIAGRAM
NOTE.
THE TIE RODS ARE NOW GENERALLY REPLACED BY BARS. SEE TEXT.
SHOE AT FIXED END.
COTTER.
GIBB.
PIPE SEPARATING PIECES
FORKED EYE
¼" PACKING PIECES
55 FT. SPAN
SCALE FOR TRUSS
SCALE FOR DETAILS.

Fig. 142.

end of Chapter VIII. the safe load = 2.4 tons per square inch, giving a total safe load of $2.4 \times 4.25 = 10.2$ tons.

By Rankin's formula the breaking load = 53 tons.

When calculating the strength of a compression member in a roof truss it is unnecessary to use such a high factor of safety as would be employed in designing a pillar to support a portion of a building. The selected section may therefore be considered sufficiently strong.

Struts are usually formed of T-iron, angle iron, or of two bars bolted together with distance pieces between, as is shown in Fig. 142. For member OP, which is 7 feet long, two bars $3 \times \frac{5}{16}$ inches may be selected, which, allowing for $\frac{3}{4}$-inch bolt holes, give 1.4 square inch sectioned area. The approximate value of $r = .29D = .29 \times 3 = .87$. $\therefore \frac{l}{r} = 94$. And according to table at end of Chapter VIII., the safe load per square inch = 2.6 tons, or a total load of $2.6 \times 1.4 = 3.64$ tons = 73 cwts. The

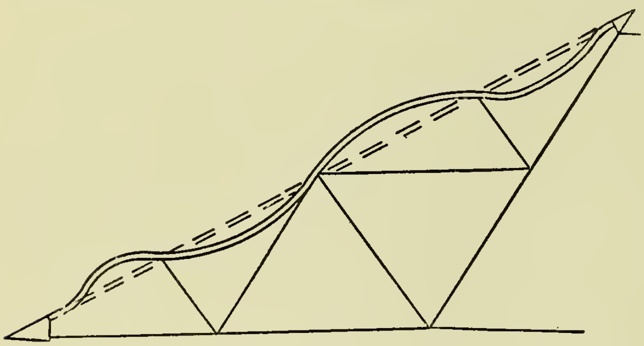


Fig. 143.

stress in this member is $70\frac{1}{2}$ cwts.; therefore the member is strong enough.

Member MN, formed of two – $2 + \frac{5}{16}$-inch bars, is amply strong; but for practical reasons cannot be made of lighter section.

It should be observed that the sizes of members given in this and following examples are the least that should be allowed, but it may often be advisable to use sections heavier than the minimum in order to secure greater uniformity, and to avoid delay in construction.

In drawing the elevation of the truss, lines as shown upon the frame diagram should first be laid down. The members are then drawn, treating the original lines as centre lines, or lines drawn through the centres of gravity of the section of each particular member. If this precaution be not taken, secondary stresses may be produced which will not have been allowed for in designing the members.

Joints.—Every joint should be so arranged that each member coming upon the bolt or rivet puts it into double shear. It is for this purpose that members NO and PQ have been given forked extremities.

Had the lower end of member NO been made with a single eye coming upon the bolt at one side of the joint with the strut on the opposite side, as in Fig. 144, a twisting moment would have been produced as indicated in the figure, which could only be resisted by pressure upon the nut and head of the bolt.

These forked ends are difficult to form accurately, and their use should be avoided as far as possible. This may sometimes be done by the use of heavier bolts,

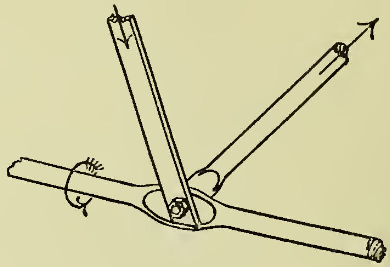


Fig. 144.

provided that the secondary stresses produced are thoroughly considered. It should always be borne in mind that although as a general rule, for the sake of economy, as little metal as possible should be used in the design of every structure, yet by the use of apparently unnecessary metal much difficult labour may often be

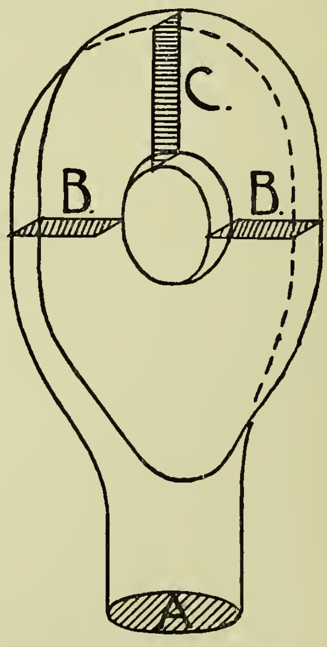


Fig. 145.

saved, thereby considerably reducing the actual cost of the structure.

In forming the eyes of rods (Fig. 145), the sectional area of metal on either side of the hole should approximately be equal to one and a quarter times the area of rod. Thus B + B = 1.25 A, while area C = area A. If the eye be forked at least half the amount should be allowed in either half of the fork.

In calculating the necessary size of bolts to connect members together, a reduction of about 20 per cent. of the safe stresses should be made, as bolts do not completely fill the hole in the way that rivets do. Safe shear stress may therefore be taken at 4 tons per square inch, and the safe bearing stress at 7 tons per square inch. Taking double shear as 1.75 time single shear, it is found that a bolt of $1\frac{1}{4}$-inch diameter is necessary to connect member RS to the head of the truss, and to resist the shear of 146 cwts. The joint will also be safe against bearing; however, were the packing pieces shown in the illustration not employed, the eye of member RS would be only $\frac{1}{2}$ inch thick, in which case the diameter of the bolt would need to be considerably increased to obtain sufficient area to resist bearing.

To connect the T to the $\frac{1}{2}$-inch plates at head of truss, sufficient rivets must be used to resist a shearing and bearing stress of 214 cwts. Three $\frac{3}{4}$-inch rivets will be found just enough.

Considering the joint at the head of strut QR, a $\frac{5}{8}$-inch bolt would be strong enough to resist the thrust of $35\frac{1}{2}$ cwts. offered by the strut, but practically a $\frac{3}{4}$-inch bolt is the smallest that should be used in a structure.

The shear stress which the bolt at the head of strut OP has to resist is not as clear as in the cases taken above. The thrusts upon the bolt at this joint are shown at A, Fig. 147. The components of each thrust, parallel and at right angles to the thrust of the rafter, are shown in dotted lines. The bolt may be considered as a beam with thrusts acting upon it at the central points in the thickness of each member. At B the components parallel to the rafter of the forces acting upon the bolt are shown in their respective positions, the thrust of each member being divided in two by the fork at its extremity. In the same diagram is shown the shear stress caused by these thrusts, found as described in Chapter III. At C the components acting upon the bolt at right angles to the rafter are shown, together with the shear stresses caused by them. Comparing these two shear stress diagrams, it is seen that the maximum stress occurs between members ON and QP, where there is a stress of $20\frac{1}{4}$ cwts. acting parallel to the rafter and $8\frac{5}{8}$ cwts. acting at right angles to the rafter. These two shear stresses form the components of the maximum shear, and the resultant stress = 22 cwts., as shown at D.

E and F show the effect of arranging the members differently, the two parts of the strut PO being here shown acting at the ends of the bolt. The result is seen to be a large increase in the stresses at right angles to the rafter, while tendency to bending would be similarly increased. The maximum shear stress is now seen to be $35\frac{1}{4}$ cwts., while the stress between members ON and QP has been increased to $33\frac{1}{2}$ cwts., as shown at G. This demonstrates the importance of arranging members on a bolt so that the thrusts act as far as possible alternately in opposite directions, and with the greatest thrusts as near as possible to the centre of the bolt.

It may be thought that too much has been said about the design of a single small bolt, but it has been given here merely to demonstrate the principles which govern the strength of bolts in similar positions,—and this is an important matter, inasmuch as a single weak connection may render an otherwise well designed roof quite useless.

The maximum shear stress in the bolt under consideration being 22 cwts., a bolt $\frac{5}{8}$-inch diameter would be large enough to resist this; but in order to obtain sufficient bearing surface to resist the thrust of $70\frac{1}{2}$ cwts. given by the strut, a $\frac{7}{8}$-inch bolt must be used. The resistance to bearing here will then be $2 \times \frac{7}{8} \times \frac{5}{16} \times 7$ tons = 3.8 tons, or 76 cwts.

The stresses acting upon the bolts at the feet of struts MN, OP, and QR may be found by the methods above described.

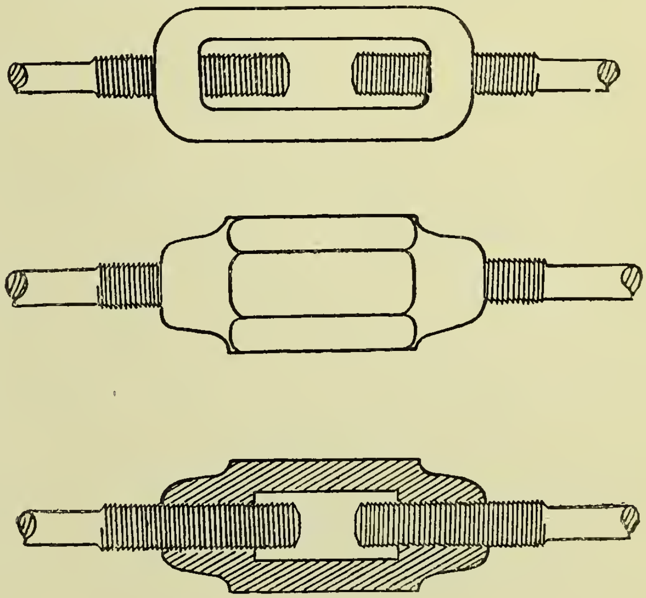


Fig. 146.

Some method of tightening up the tie rods of trusses is generally employed. In the present case (Fig. 142) the tightening is effected by means of a gib and cotter connection, as shown in the detail of the shoe. A similar result may be effected by means of "turnbuckles" (Fig. 146). This provision is to cover the effect of defective workmanship, and would not be required if the work were accurately carried out. In the very general practice now employed of using flats and angles in place of rods, and rivets in place of bolts, provision for tightening is seldom if ever made.

Fig. 148 shows a design for a truss of equal span and spacing, and with the same loading as that assumed in Fig. 142.

It will be noticed that members MN and YZ are unstressed, and might therefore be omitted; they, however, help to support a portion of the tie rod, and rather improve the appearance of the truss.

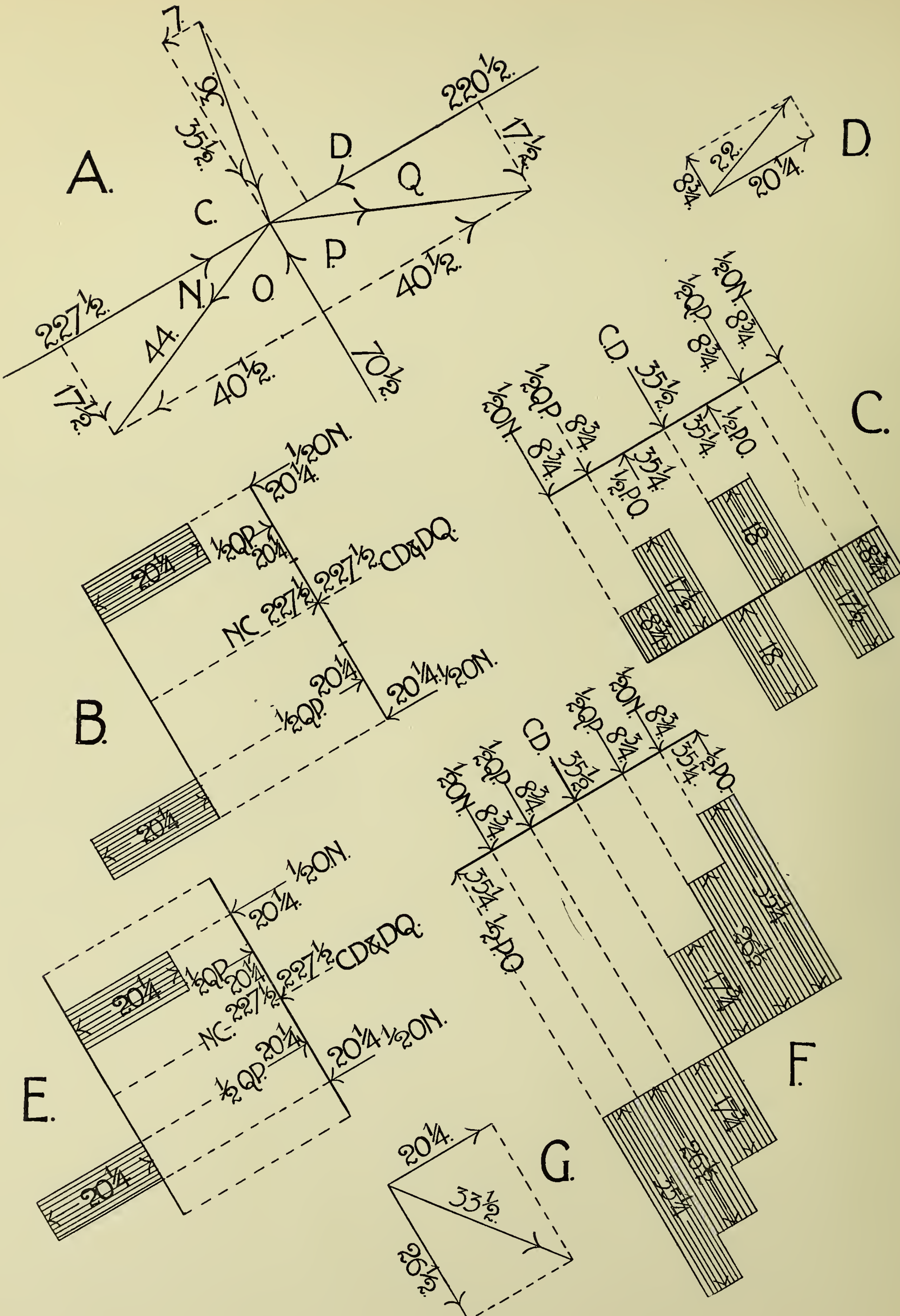

A.
B.
C.
D.
E.
F.
G.
7.
36.
35½.
220½.
17½.
Q
P.
O.
N.
227½.
44.
40½.
70½.
22.
8¾.
20¼.
½ON.
½QP.
CD.
½PO.
35¼.
18.
NC. 227½.
227½. CD&DQ.
26½.
17¼.
33½.

Fig. 147.

STEEL ROOF TRUSS. (ROD FORM.)
55 FT. SPAN. 10 FT. BETWEEN CENTRES.
DEAD LOAD 19. LBS. PER FT. SUPER.
NORMAL WIND PRESSURE 33 LBS. PER FT. SUPER.

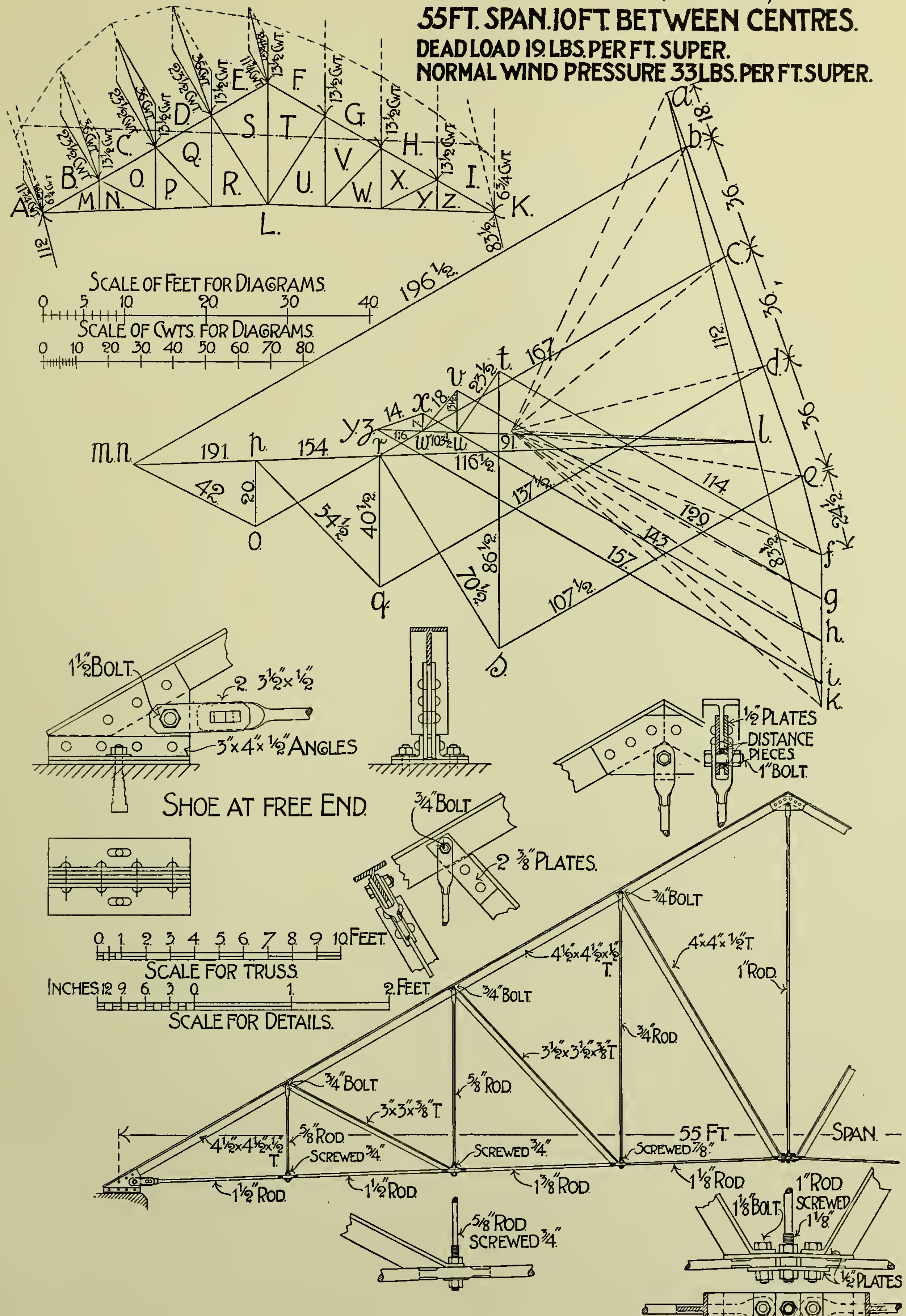


FIG. 148.

The stresses in the members, as found by the diagram and the various sections employed, are given in the following table :—

Member.	Tension Cwts.	Compression Cwts.	Necessary Sectional Area.	Form of Member.
ML or NL, and YL or ZL .	191		1.59	$1\frac{1}{2}''$ Rod.
PL and WL . .	154		1.28	$1\frac{3}{8}''$,,
RL and UL . .	$116\frac{1}{2}$		0.97	$1\frac{1}{8}''$,,
ST	$86\frac{1}{2}$		0.72	$1''$,,
QR and VU .	$40\frac{1}{2}$		0.33	$\frac{3}{4}''$,,
OP and XW .	20		0.17	$\frac{5}{8}''$,,
MN and ZY .	0		0	$\frac{5}{8}''$,,
BM and IZ . .		$196\frac{1}{2}$	Length 8′ 0″	$4\frac{1}{2}''\times4\frac{1}{2}''\times\frac{1}{2}''$ T.
CO and HX .		167		
DQ and GV .		$137\frac{1}{2}$		
ES and FT . .		$107\frac{1}{2}$		
RS and UT . .		$70\frac{1}{2}$	13′ 0″	$4''\times4''\times\frac{1}{2}''$ T.
PQ and WV .		$54\frac{1}{2}$	10′ 0″	$3\frac{1}{2}''\times3\frac{1}{2}''\times\frac{3}{8}''$ T.
NO and YX . .		42	7′ 9″	$3''\times3''\times\frac{3}{8}''$ T.

It will be observed that tee steel has been used for the struts, which are made to exert an even thrust upon the bolts at their heads by the use of plates riveted on either side of their vertical limbs.

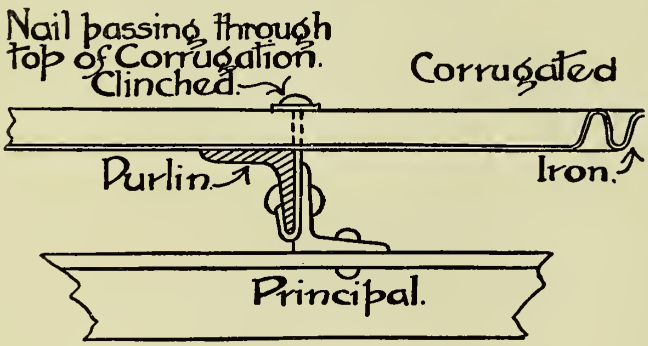


Fig. 149.

To form the joint at foot of strut, the vertical limb is here cut away, while its table is forged at an angle. An eye is formed in the horizontal tie rod by means of a weld, while the whole is fastened together by nuts on the threaded end of the vertical member. This threaded end must have a "plus thread,"—that is to say, the diameter at the thread's bottom must be *at least* as large as the diameter of the rest of the rod. This is effected by "upsetting" or "jumping up" the end of the rod, after which the thread is cut upon it. In all cases where threads are cut upon the members of a truss they must be plus threads.

Plate IV. shows a design for a steel roof truss of 90 feet span spaced 12 feet apart.

The covering is assumed to be corrugated iron, which may if desired be fixed direct to steel purlins as indicated in Fig. 149. Corrugated iron left bare on the under side causes condensation upon its surface, the dripping from which may give considerable inconvenience. For this reason the corrugated iron is shown in Plate IV. to be laid upon boarding.

The length of rafter = 52 feet, and the total roof area supported by a single truss = 52 × 12 × 2 = 1248 square ft.

It may be assumed that the truss will weigh 8000 lbs., or say $6\frac{1}{2}$ lbs. per square foot. The detailing of the dead loading is then as follows :—

Truss . . .	$6\frac{1}{2}$	lbs. per square foot.	
Purlin	2	,,	,,
Boarding . . .	$2\frac{1}{2}$	,,	,,
Corrugated iron .	2	,,	,,
Total dead load	13	,,	,,

Normal wind pressure, 33 lbs. per square foot.

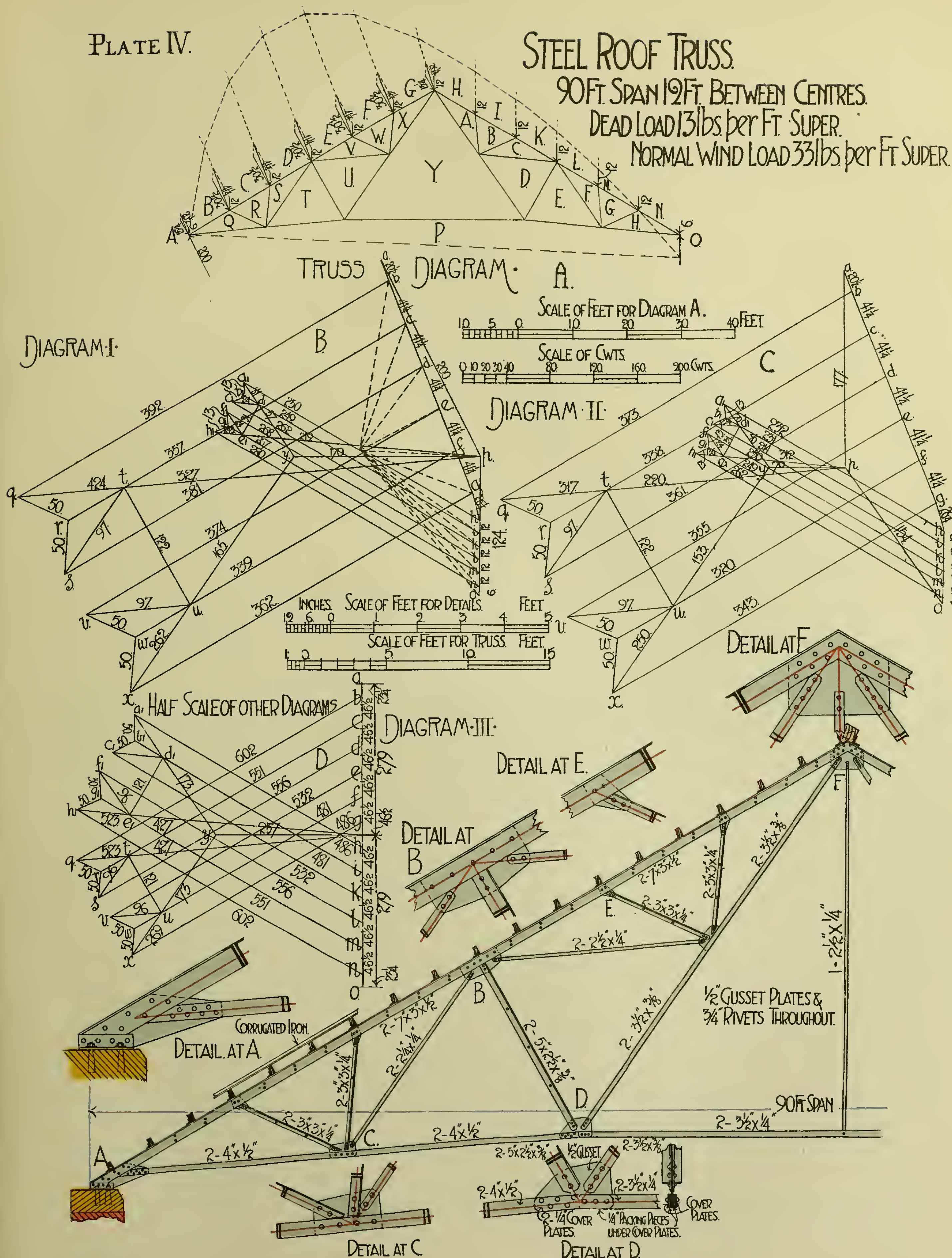
Although the loading is seen to be distributed by the purlins throughout the length of the principal rafter, yet, in so far as it affects the trussing in general, it may be considered as acting at the joints between principal rafter and struts, as shown at A, Plate IV. At A and B the reactions have been found, as described previously, considering that the right-hand end of the truss is supported upon rollers, and the stress diagram has been proceeded with as usual. At C the stress diagram is given for the case when the wind blows upon the "free" end of the roof; but instead of altering the direction of the loads upon the diagram the left-hand extremity is now considered as the "free" end. This expedient makes the drawing of the diagram simpler and the variation of stress more apparent. The polar and funicular diagrams have again to be drawn for the latter case, but they have here been omitted to avoid confusion.

The stresses in the various members found by these diagrams are tabulated below, and it will be noted that, in the particular case which is now being considered, the maximum stress in every member is greatest when the wind is blowing from the fixed end of the truss; but it must not be concluded that this will always be the case.

It is a very usual practice, in working out the stresses in a truss, to assume a uniform vertical loading, usually 60 lbs. per square foot of roof surface, this being intended to cover the effect of all dead and live loads. The stress diagram for the same truss, but with a vertical load of 50 lbs. per square foot, is shown to a smaller scale at D. The stresses thus found are given in the following table, and it will be noticed that those produced in the struts and minor ties are almost identical with the stresses found by the more exact method; but that the stresses in the rafter and main tie are considerably more when found by the last method.

In calculating the size of the members in tension a stress of 7 tons to the square inch has been allowed. Additional metal must be allowed in each member to compensate for the removal of metal for one rivet hole. A light vertical bar is shown attached to the apex plate, whose sole duty is to support the central portion of the tie bar, which without this would be of too great a span. This member has not been indicated upon the truss diagrams, as it does not affect the framing.

PLATE IV.


STEEL ROOF TRUSS.
90 FT. SPAN 12 FT. BETWEEN CENTRES.
DEAD LOAD 13 lbs. per FT. SUPER.
NORMAL WIND LOAD 33 lbs. per FT. SUPER.
TRUSS DIAGRAM. A.
SCALE OF FEET FOR DIAGRAM A.
SCALE OF CWTS.
DIAGRAM I.
DIAGRAM II.
DIAGRAM III.
HALF SCALE OF OTHER DIAGRAMS
INCHES. SCALE OF FEET FOR DETAILS. FEET.
SCALE OF FEET FOR TRUSS. FEET.
DETAIL AT F.
DETAIL AT E.
DETAIL AT B.
DETAIL AT A.
CORRUGATED IRON.
1/2" GUSSET PLATES & 3/4" RIVETS THROUGHOUT.
90 FT. SPAN
1/2" GUSSET
COVER PLATES.
2-1/4" COVER PLATES.
1/4" PACKING PIECES UNDER COVER PLATES.
DETAIL AT C
DETAIL AT D.

The necessary sizes of struts have been calculated as previously described.

The calculations necessary to arrive at the size of the rafters need further consideration, for in this case they have to resist a bending moment *as well as* the compressive stress found by the stress diagram. Each portion of rafter may here be considered as a pillar with fixed ends, for not only are the joints more rigid than in the cases previously considered, but the loads brought to bear by the purlins between the joints will prevent any upward deflection. (See Fig. 150, which may be compared with Figs. 98 and 143.) Begin by assuming that two angles 7 × 3 inches will probably be suitable. The approximate value of $r = .2\ D = 1.4$ inch.

$$\therefore \frac{l}{r} = \frac{8 \text{ ft. } 8 \text{ ins.}}{1.4 \text{ in.}} = 74,$$ and by the table at the end of Chapter VIII. the safe stress per square inch = 4.6 tons.

∴ Area necessary to resist the compressive stress $= \frac{392}{92} = 4.3$ square inches.

Member.	Maximum Stress in Cwts.						Necessary Sectional Area.	Form of Members.
	Wind upon Fixed End.		Wind upon Free End.		Vertical Load 50 Lbs. per Sq. Ft.			
	Tens.	Comp.	Tens.	Comp.	Tens.	Comp.		
QP and H_1P .	424		317		523		3.03	2–4″ × ½″ Flats.
TP and E_1P .	327		220		427		2.34	
PY	170		70		257		1.21	3½″ × ¼″ ,,
XY and A_1Y .	262		250		269		1.87	2–3½″ × ⅜″ ,,
UY and D_1Y .	165		153		173		1.17	
ST and F_1E_1 .	97		97		96		0.69	2–2½″ × ¼″ ,,
UV and D_1C_1 .	97		97		96		0.69	
BQ and NH_1 .		392		373		602	Length 8′ 8″	2–7″ × 3″ × ½″ L.
CR and MG_1 .		357		338		551		
DS and LF_1 .		381		361		556		
EV and KC_1 .		374		355		532		
FW and IB_1 .		339		320		481		
GX and HA_1 .		362		343		486		
TU and E_1D_1 .		122		122		121	12′ 0″	2–5″ × 2½″ × ⅜″ L.
QR and H_1G_1 .		50		50		50	7′ 6″	2–3″ × 3″ × ¼″ L.
RS and G_1F_1 .		50		50		50		
VW and C_1B_1 .		50		50		50		
WX and B_1A_1 .		50		50		50		

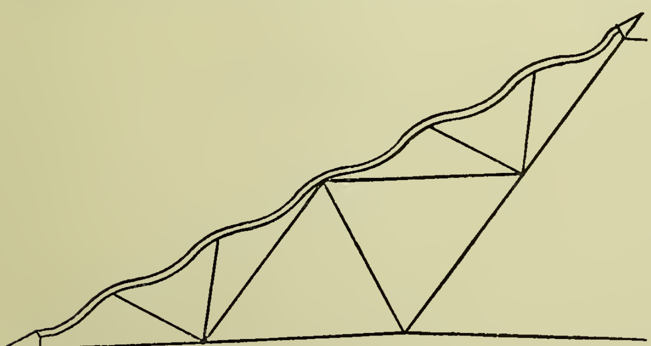


Fig. 150.

On examining the bending moment in the rafter it is at once apparent that this is a form of continuous beam, and it might be maintained that the loads assumed at the joints of the structure should be divided up in the same proportion as are the reactions in Fig. 70; but the structure is not perfectly rigid, and it will probably be just as near to the truth if it be assumed that the loads are apportioned equally, as has been done. It is only necessary to consider the span or portion of rafter at the lowest end, for it is here that the greatest combined effect of compression and bending moment will take place. The result will then be on the safe side, if it be assumed that the bending moment is equal to ⅔ of the BM for a single span similarly loaded. The BM will then be $\frac{2}{3} \times \frac{41}{3} \times \frac{8 \text{ ft. } 8 \text{ ins.}}{3} = 316$ inch-cwts.; and, putting 316 in the place of Wd in the formula given at the end of Chapter IX., $A = \frac{316y}{r^2 f}$. Then, assuming that $y = 4\frac{3}{8}$ inches, and $f = 7$ tons, or 140 cwts.—

$$A = \frac{316 \times 4\frac{3}{8}}{(1.4)^2 140} = 5 \text{ square inches} = \text{area necessary to resist BM.}$$

∴ Total area necessary to resist compression and BM = 4.3 + 5 = 9.3 square inches.

Two angles 7 × 3 × ½ inches have an area of 9½ square inches, and these will therefore, according to the above calculations, be very suitable.

It will be noticed that joint D has been formed with cover plates in addition to the gusset. These are necessary to ensure an even pull, as the two portions of the tie bar come upon the gusset at its very edge.

The number of rivets at each joint are proportioned to resist shear and bending stress, as previously explained, except in certain positions where the number is increased for the sake of greater rigidity.

This form of roof construction is now almost universal. It has the objection that additional metal must be allowed throughout the whole length of all ties to compensate for metal removed in rivet holes, but against this may be put the greater reliability and consequently higher safe stress per square inch that may be allowed as compared with rods with welded lines between each pair of joints, as has been done, Fig. 152, which is the frame diagram for this truss. Member HG may be considered as being terminated at its left end at the point where it is connected to the web; the line of direction of this member is then produced until it cuts the centre line of the rafter at point Y. The portion of truss between points Y and Y_1 may be considered as a distinct truss, and the stress diagram can be drawn in the ordinary way as in Fig. 152, totally disregarding all loads and portions of truss outside these points.

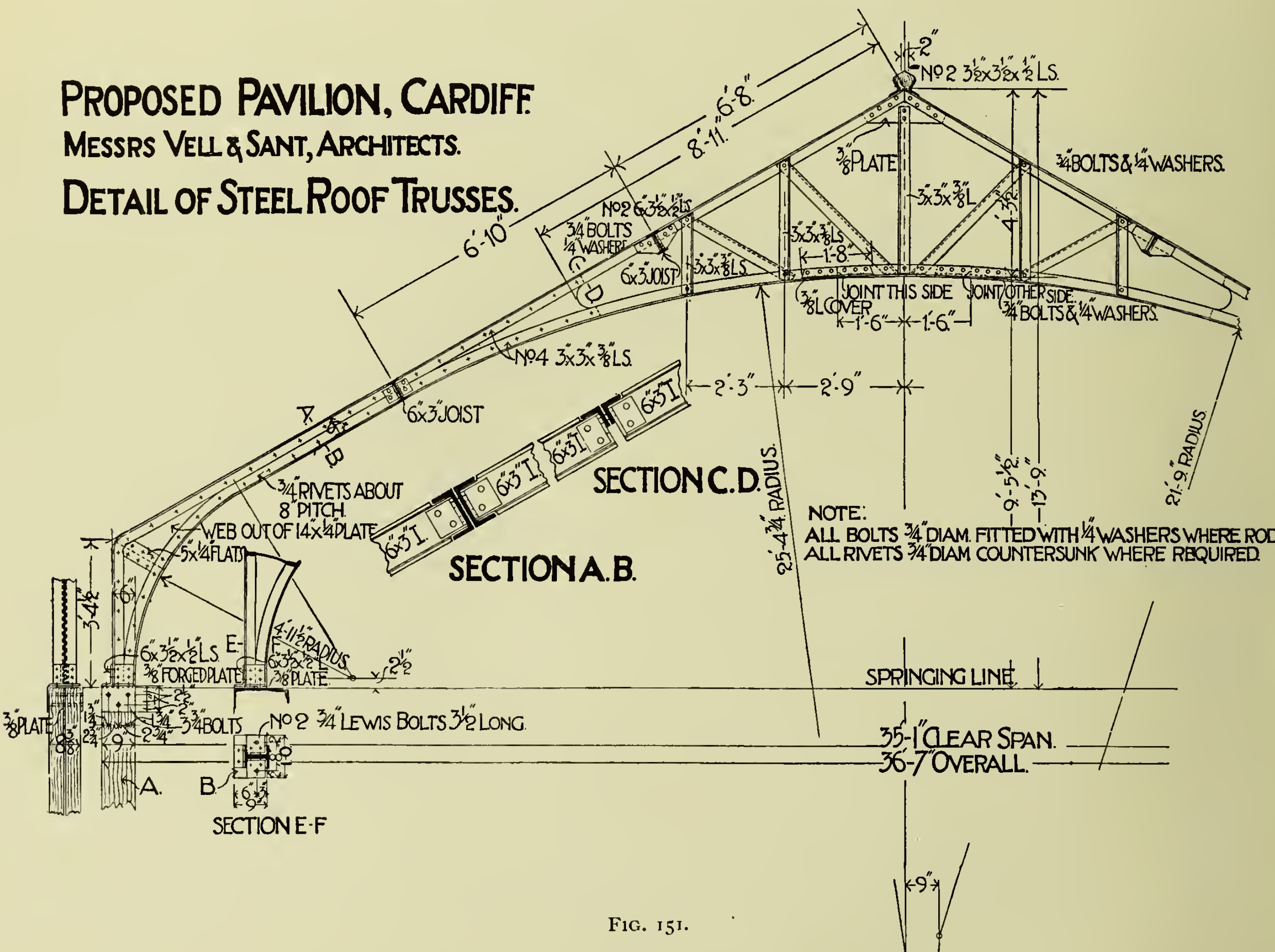


Fig. 151.

joints; while the great advantage in the use of flats in place of rods is the ease and economy in forming the joints.

Again, the danger from "invisible" flaws in the ties, as illustrated in the Charing Cross disaster, is provided for by the use of double tie bars, while this again has the disadvantage that the surfaces between the double members cannot easily be cleaned and painted.

Fig. 151 shows a roof truss by Messrs. Dawnay & Sons, which presents several distinctive features.

Whenever a curved member is met with in constructing a diagram it may be represented by straight

The curved tie IG, LG, etc. must be made of such proportions that it will resist not only the tensile stress, but also the BM which tends to straighten the member. This BM = the tensile stress × the deflection midway between the joints.

The portion of the structure beneath W_1 is evidently a simple type of plate girder, and should be designed as explained in Chapter VII., the maximum bending moment being equal to $R_1 \times X$.

The use of rolled steel joists to form the purlins and their attachments to the truss are worthy of notice.

ARCHED ROOF TRUSSES may be either three-hinged or two-hinged arches.

Three-hinged Arch.—Fig. 153 illustrates diagrammatically a three-hinged arch. Each half of the arch may be considered as a simple truss, consisting of a girder supported at one end upon an abutment and at the other end by its fellow truss ; that is to say, by the other half of the arch. Consider first one-half of the arch loaded vertically as shown. The vertical reactions R_1 and R_2 at its end are found by the usual polar and funicular diagrams. Now, reaction R_2 evidently cannot be a single force, and must be met by two forces

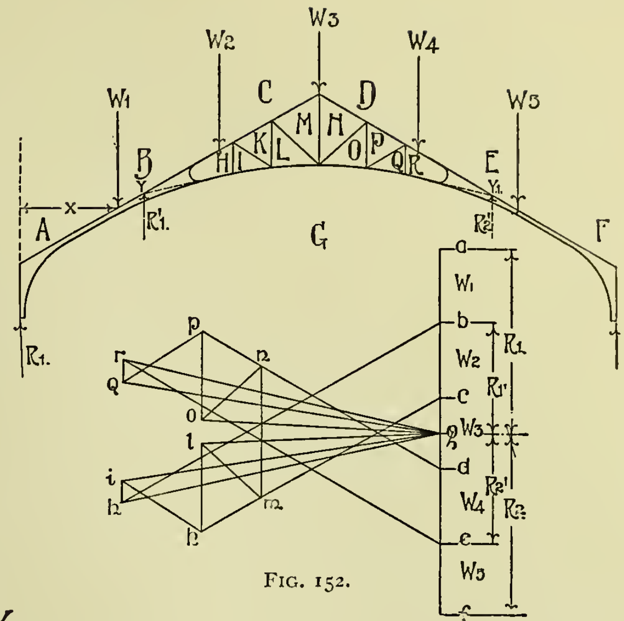


FIG. 152.

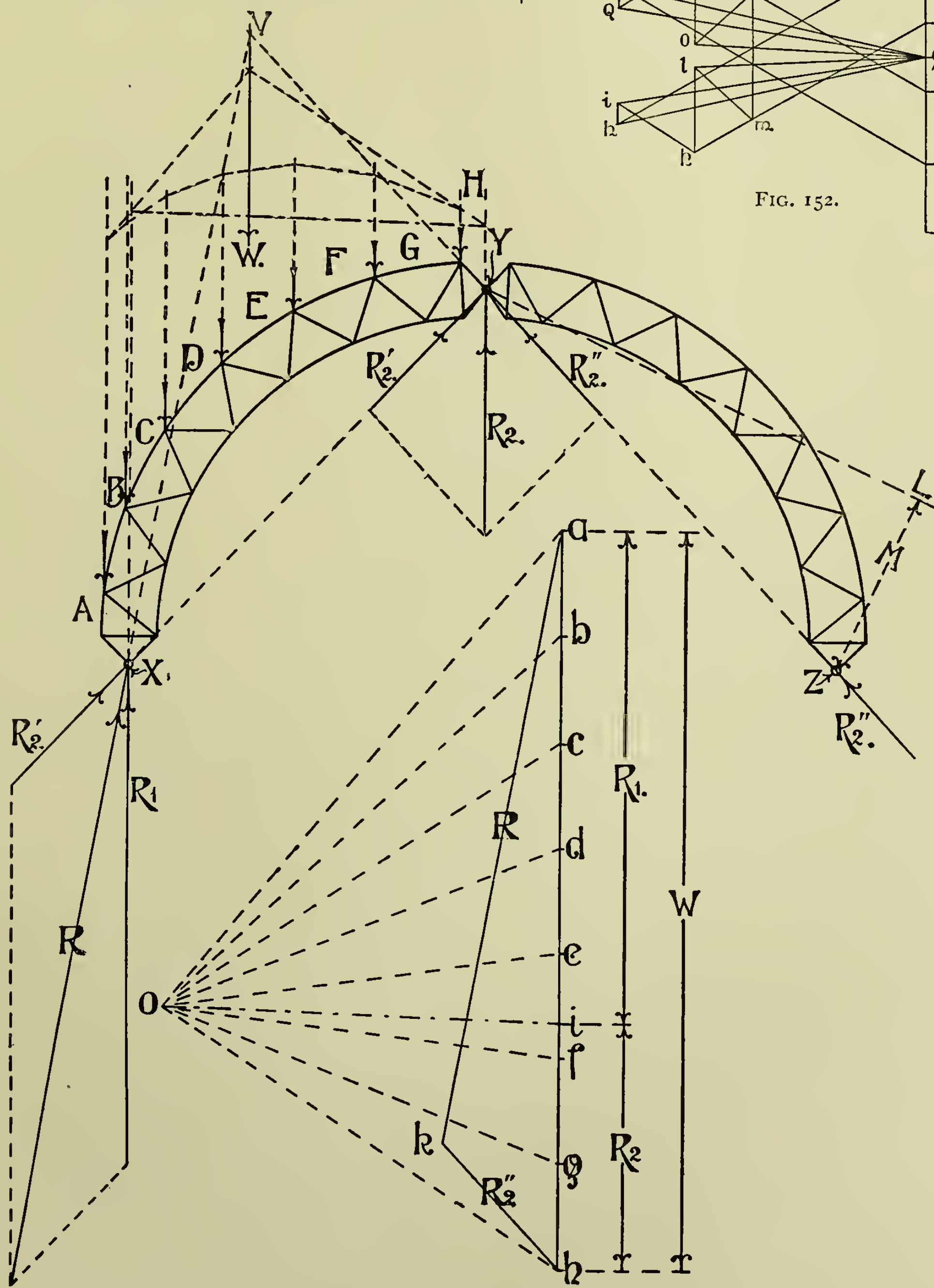


FIG. 153.

exerted by the two halves of the arch. These forces must each be in the direction of straight lines passing through the hinges at either end of the trusses, for otherwise a turning moment will be introduced as indicated at LM, and the arch would be then unstable. These components of R_2 are shown at R_2' and R_2'', and funicular diagram, the position of the resultant load W is found. As already observed, the right-hand reaction must pass through the two hinges Y and Z. Drawing a line through these two points and producing it, it will cut the line of direction of W at V ; and by joining XV, the direction of the left-hand reaction is found.

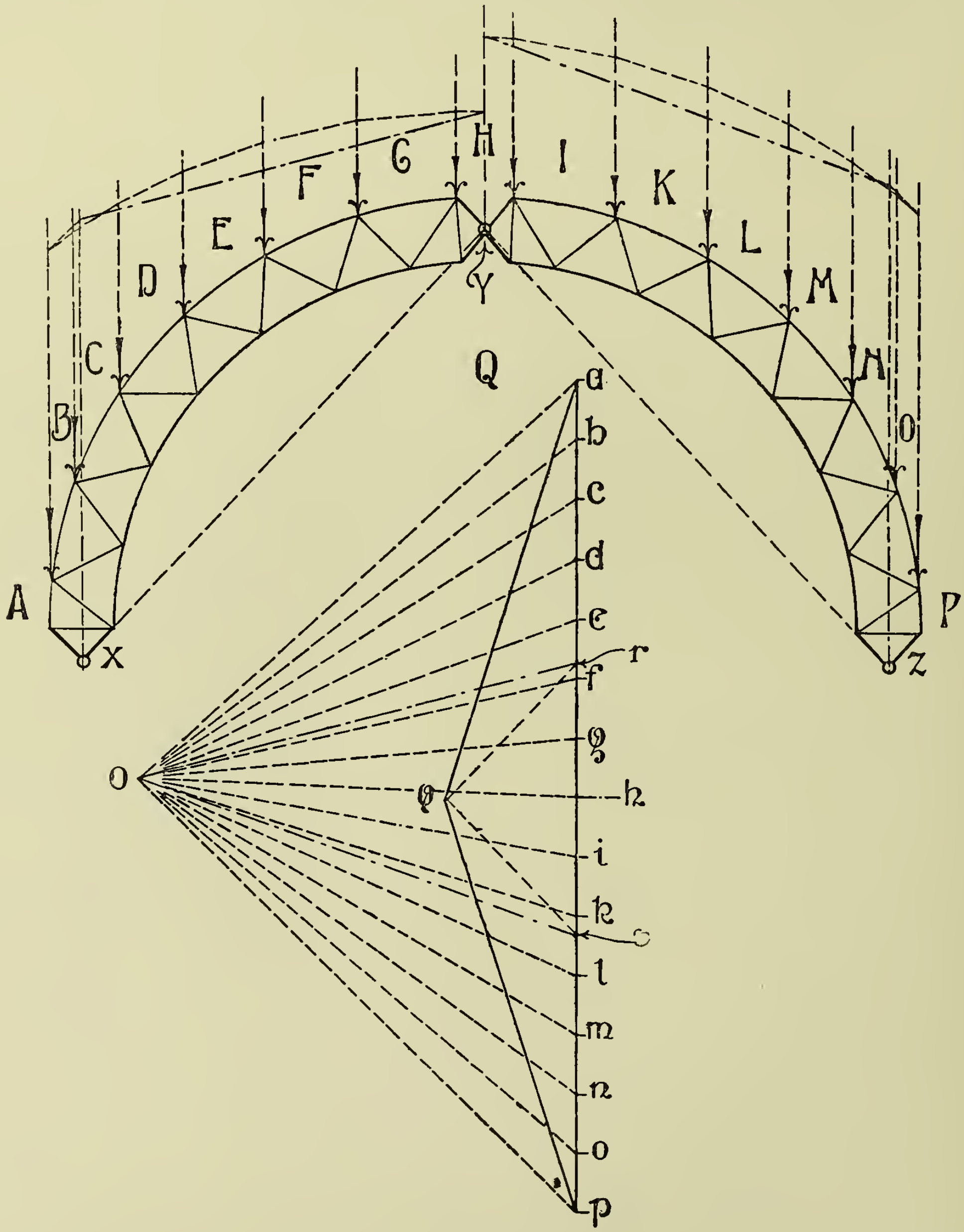


Fig. 154.

form part of the total reactions at the abutments. Then the total reaction at the left-hand abutment due to the loading on the half of the arch under consideration is given by the resultant of R_1 and R_2'.

The reactions at the abutments may be found also by the principle stated at the start of Chapter I.,—namely, that the lines of direction of three forces in equilibrium must meet in a point. By drawing the lines shown dotted parallel to *oa* and *oh* upon the The amounts of the reactions may now be ascertained by the triangle of forces *ahk* (Fig. 153).

The reactions, as found above, are due only to the loading of the left-hand half of the arch. The right-hand half will produce similar reactions, which must be compounded with those already found in order to arrive at the total reactions of the abutments. The investigation of both halves may be simply combined in one diagram, as shown in Fig. 154. In drawing the

polar diagrams, only one pole O has been used for the two halves of the arch. Thus *ar* and *rh* are found to be the vertical reactions for the left-hand half, and *hs* and *sp* those for the right-hand half of the arch. Thus forces a total reaction *qa*. The reaction *pq* is found similarly.

The same case may be considered in a slightly different light. Having found by the polar construction

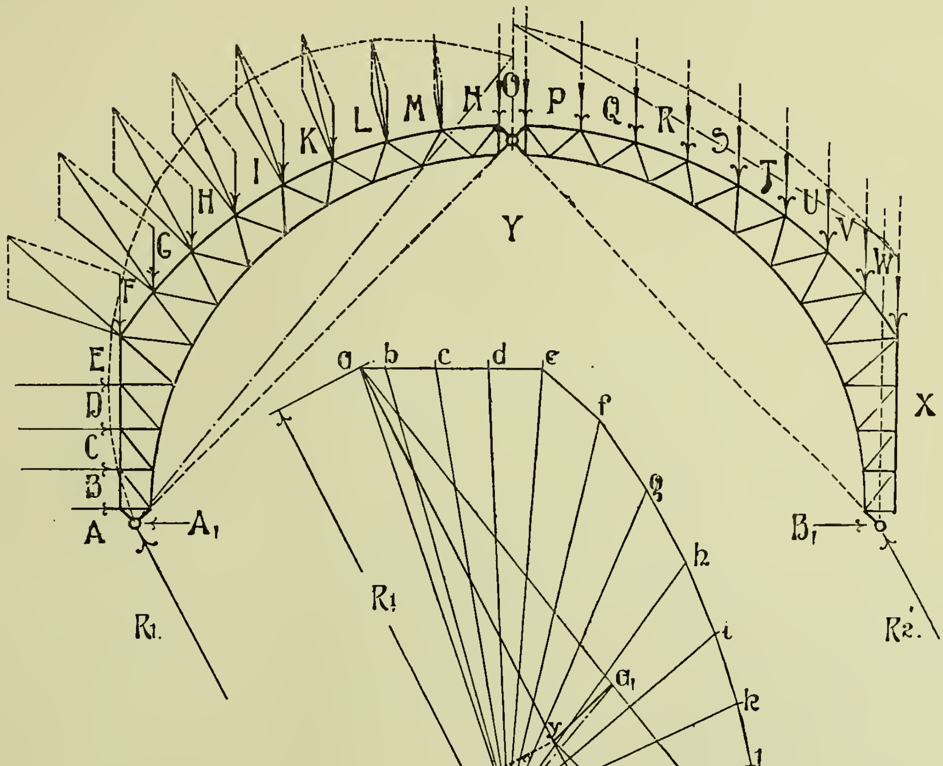
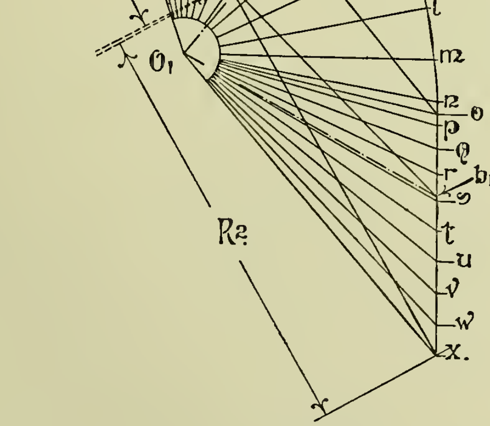



FIG. 156.

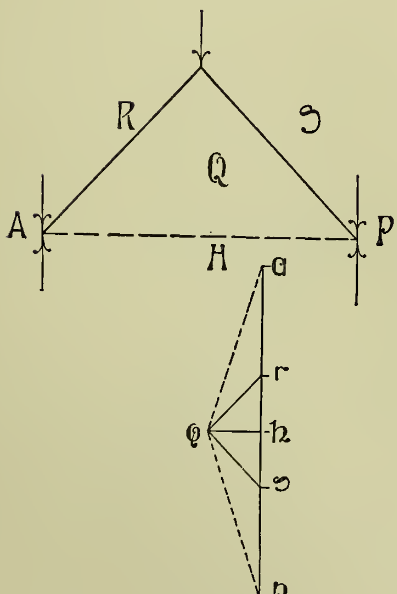


FIG. 155.

rs = the vertical reaction at the central hinge, while *rq* and *qs*, drawn parallel to XY and YZ, give the amounts of the components of this vertical force, and these components form part of the reactions at the abutments. Then, at the left-hand abutment there are the vertical force *ar* and the thrust *rq*, giving by the triangle of

as above that the vertical forces at the three hinges are respectively equal to *ar*, *rs*, and *sp*, each half of the arch may be considered as a single member of a simple truss, and, as stated previously, such curved members may be represented upon the diagrams by straight lines between the joints, as RQ and QS in Fig. 155, where an imaginary tie has been inserted to take the horizontal thrust. On account of this imaginary tie the reactions may for the present be considered to be vertical, and the stress diagram may be drawn as usual, when the stress in the tie is found equal to *qh*; but this thrust is in reality taken up by the abutment. Thus at the left-hand abutment there is a horizontal

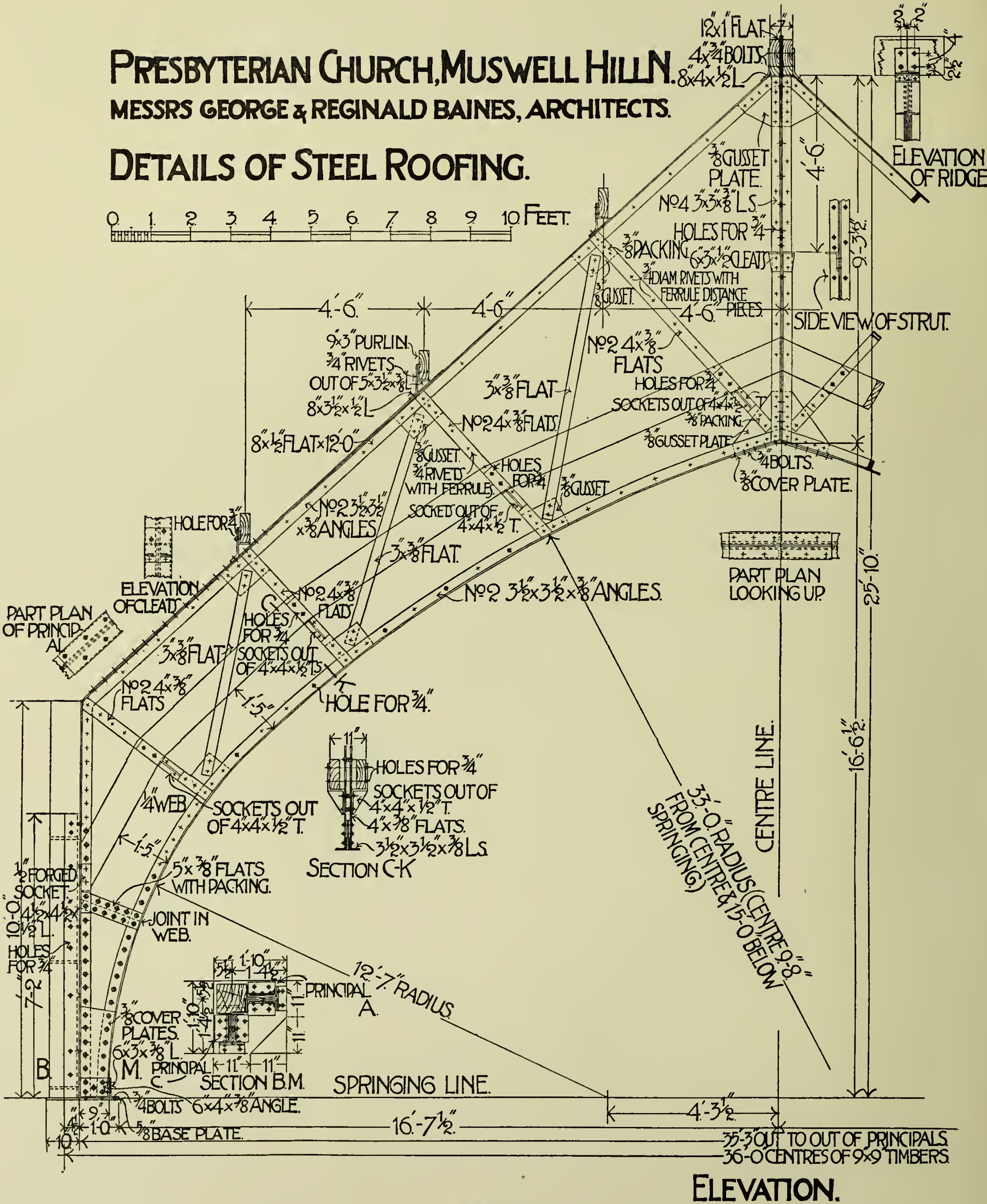

PRESBYTERIAN CHURCH, MUSWELL HILL N.
MESSRS GEORGE & REGINALD BAINES, ARCHITECTS.
DETAILS OF STEEL ROOFING.
0 1 2 3 4 5 6 7 8 9 10 FEET
ELEVATION OF RIDGE
SIDE VIEW OF STRUT
PART PLAN LOOKING UP
ELEVATION OF CLEAT
PART PLAN OF PRINCIPAL
SECTION C-K
SECTION B.M.
PRINCIPAL A.
PRINCIPAL C.
9"x3" PURLIN
8"x½" FLAT x 12'-0"
No 2 3½"x3½"x⅜" ANGLES
12'-7" RADIUS
33'-0" RADIUS (CENTRE 9'-8" FROM CENTRE & 15'-0" BELOW SPRINGING)
CENTRE LINE
SPRINGING LINE
16'-7½"
4'-3½"
35'-3" OUT TO OUT OF PRINCIPALS
36'-0" CENTRES OF 9x9 TIMBERS
25'-10"
16'-6½"
ELEVATION.

FIG. 157.

force qh, and a vertical force ha, so that the total reaction $= qa$. The stress diagram will be seen to be similar to the identically lettered portion of the diagram in Fig. 154.

Fig. 156 shows a type of large arched roof having its lower hinges on a level with the ground. A horizontal wind pressure against the vertical portion has been allowed for, but in reality this would generally be protected by a wall or building. The normal wind pressure acting at the various points on the curved surface may be obtained from the table at the end of Chapter XII. The reactions have been found as above described. The resultant of all loads upon the left-hand half of the arch is ao, and it has been divided at a_1 by means of the polar diagram, into the two parts, which may be considered as acting at the two ends. Similarly ob_1 and b_1x are the two portions of the resultant of loads upon the right-hand half, and act at its extremities. Thus, a_1ob_1 are the forces acting at

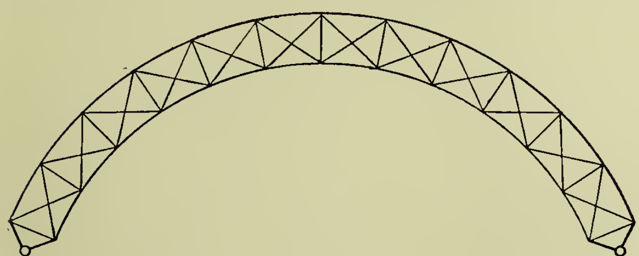


Fig. 158.

the central hinge, and they are met by b_1y and ya_1, parallel to B_1Y and YA_1. Then aa_1 and a_1y are the forces acting at A_1, giving a total resultant ya. Similarly the reaction at $B_1 = xy$. The stress diagram, which has not been given here, may be proceeded with as usual without further difficulty.

Two-hinged Arch.—Fig. 158 represents a type of two-hinged arch. The case is one which cannot be solved by any simple method, although the probable stresses in its members can be arrived at by a laborious process of trial and error; but it will not be attempted to do this here. Supposing for a moment that the ends are free, the outer member would then be in compression and the inner member in tension. This would result in the compression and extension of these members tending to slightly flatten the arch. This tendency is resisted by the hinges at the abutments. Thus the reactions and the stresses in the members depend upon the distorsion of the members, and at the same time the distorsion depends upon the stress. The case therefore can evidently only be solved by trial and error.

Fig. 157 is the detail of a roof truss by Messrs. Dawnay & Sons. Were this roof fixed firmly upon pins to abutments capable of resisting the thrust it

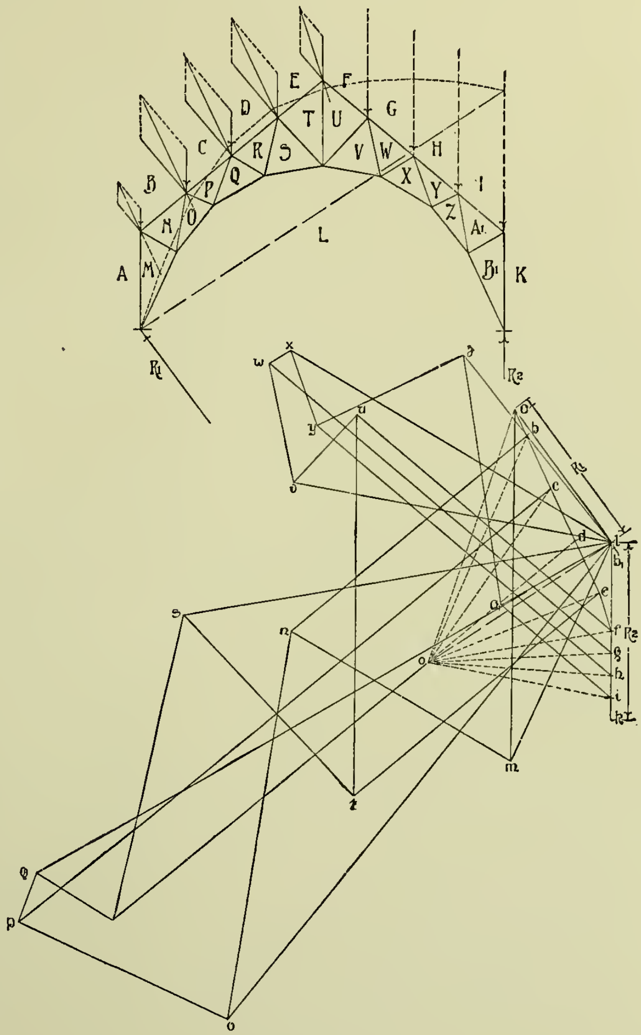


Fig. 159.

would be a typical example of a two-hinged roof arch. This is, however, not the case, and it may therefore be treated as an ordinary truss. Frame and stress diagrams are given in Fig. 159, with an assumed loading, and assuming also that the left extremity is fixed while that at the right is free.

CHAPTER XIV

THE STEEL FRAMEWORK OF BUILDINGS

The Steel Construction of Buildings has been evolved in America, chiefly for the erection of high buildings or "sky-scrapers," such as that shown in Plate V. The necessity for high buildings is purely commercial, and they are the direct outcome of the endeavour to employ expensive building land to the greatest advantage. The maximum height to which a building can be erected depends upon the total load which the plot of land is capable of carrying. In ordinary masonry construction the limit is comparatively soon reached, while the great thickness of the walls involves much loss of space on the lower floors where space is most valuable. It is obvious that the lighter the construction the greater the height that is rendered practically possible.

The steel construction of buildings has been gradually evolved. To begin with, the walls were made entirely self-supporting while the floors and their loads were carried by a metal framework of girders and stanchions, which were entirely distinct from the walls, so that the settlement, expansion, and contraction of the two parts could go on independently. This form still necessitated heavy walls.

The next step was to carry all walls upon girders at every floor level, the walls being of moderate thickness, and adding considerably to the stiffness of the structure by their weight and rigidity. This is known as SKELETON CONSTRUCTION.

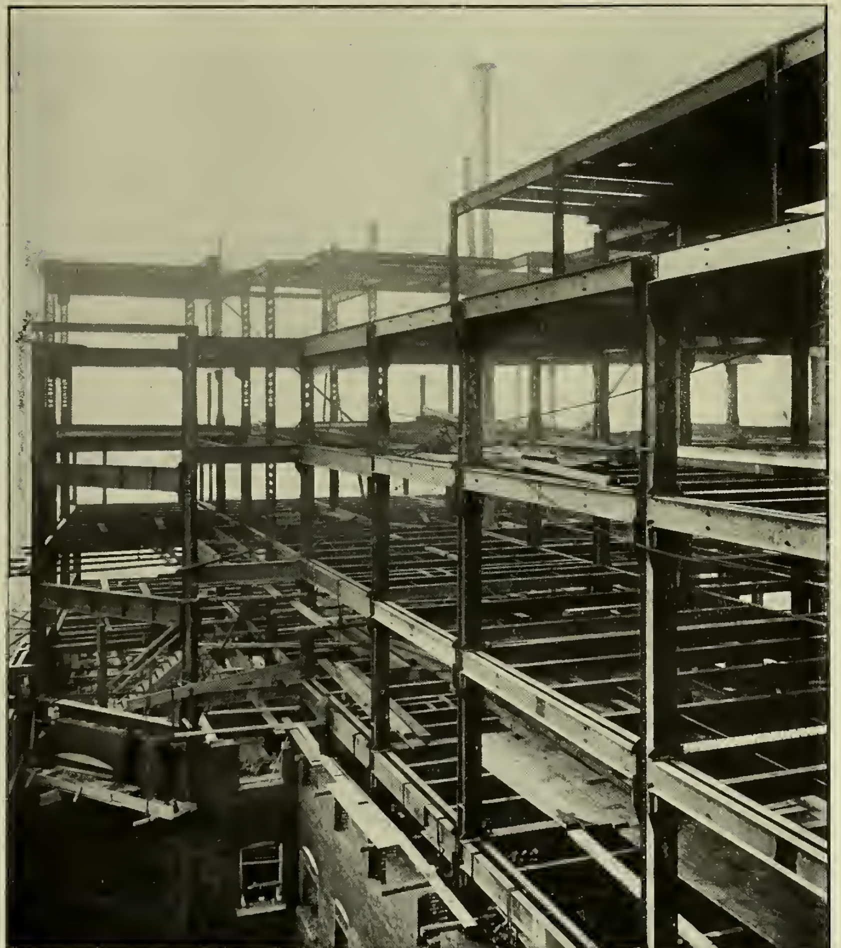
A third step was to construct the steelwork sufficiently rigidly, with the help of bracing, to resist all loads as well as wind pressures, the walls becoming simple screens, being made no thicker than is necessary for their self-support or for architectural effect. This is known as CAGE or VENEER CONSTRUCTION.

The limitations of the London Building Act will not allow the latter construction to be carried out thoroughly in London, for it stipulates that no building shall be erected of a greater height than 80 feet, exclusive of two storeys in the roof and of ornamental towers, etc. Buildings of this height may be constructed with walls of reasonable thickness, even when these walls carry the entire weight. The London Building Act lays down the thickness of external walls, and also that "Recesses and openings may be made in external walls provided, (*a*) That the backs of such recesses are not of less thickness than 8½ inches; and (*b*) That the area of such recesses and openings above the ground storey do not, taken together, exceed one-half of the whole area of the wall above the ground storey in which they are made." These regulations, under the present form of the Building Act, must be adhered to, notwithstanding that such limitations are perfectly unnecessary when all loads are carried upon a carefully framed structure of steel. However, even under the existing state of affairs, a steel framework may often be used with considerable economy, as is evidenced by the number of steel-framed structures that are now springing up; and these restrictions do not apply outside the London area. Light partitions may take the place of heavy interior walls, and the large concentrated loads which will be found in any form of building are, as a matter of course, carried by steel stanchion in place of heavy masonry piers. Probably the most thorough example of a steel-frame construction yet erected in England is that of the Ritz Hotel, Piccadilly, Messrs. Mewes & Davis being the architects, while Mr. S. Bylander of the Waring-White Building Company designed the steelwork. General views of this building are shown in Plate VI., in which the steel structure is seen partly clothed in its masonry walls.

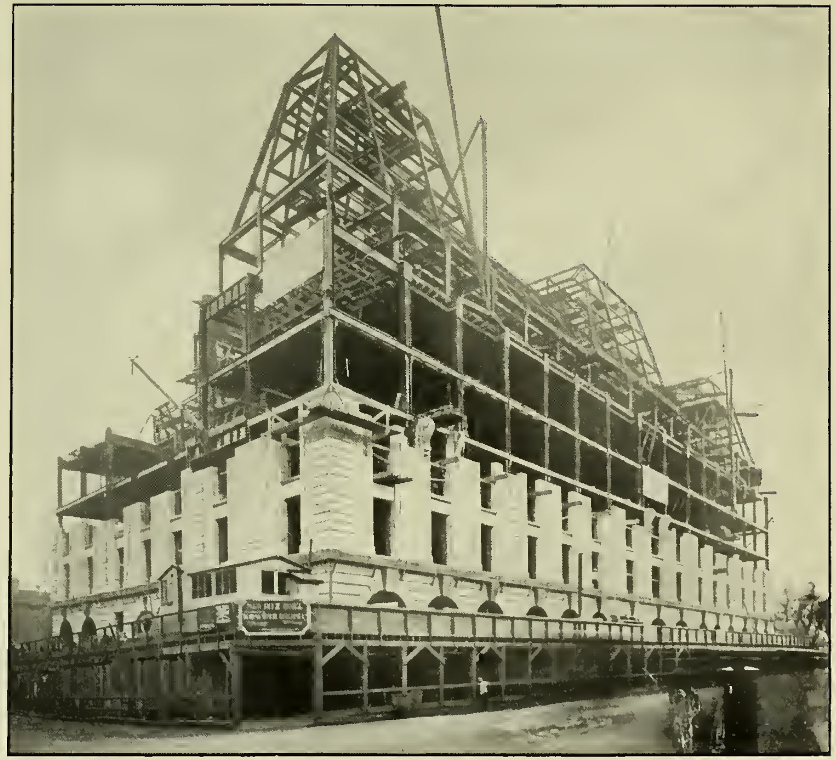
It should be observed that, in the design of steel construction it will often be more economical to apply the following principles in part only. Thus the consideration of floor beams and girders will apply equally to any masonry building in which fire-resisting floors are used. Again, the lower floors only may be formed with complete cage construction; but it must be observed that, if any girder or part of a girder be embedded in a wall, it must be designed to carry that wall, as the weight will eventually come upon it in settlement. Such cases will be best covered by the consideration of a complete example of steel frame construction.

The system consists of concentrating the weight of all floors and walls upon stanchions spaced throughout the building, by means of girders at every floor level. The resultant framework may be seen in Plate VI., while Fig. 160 gives the framing plan at the second floor level of the Ritz Hotel. It will be noticed that measurements given in Fig. 160 and in other details of this building are in metric measure, which was worked to throughout. This was necessitated by the fact that the steel was manufactured in Germany. The loads coming upon the girders which carry the walls are easily calculated, provided that the weights of the various materials are known. The following were the weights assumed in designing the steelwork of the Ritz Hotel :—

THE FULLER BUILDING, NEW YORK,
COMMONLY KNOWN AS THE FLAT IRON.

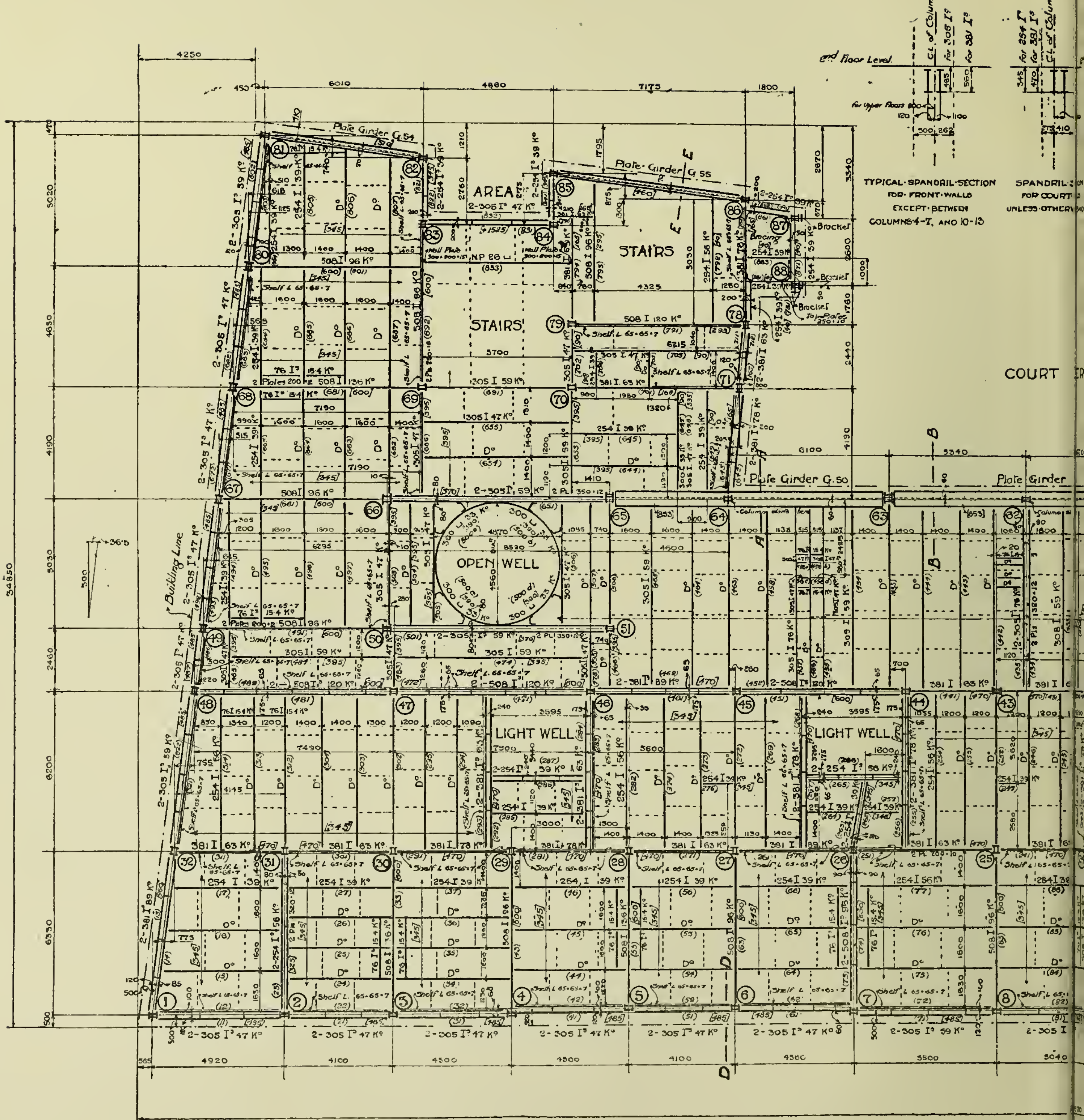


Steelwork as seen from Fourth Floor.



General View, April 1, 1905.

RITZ HOTEL, PICCADILLY, LONDON.

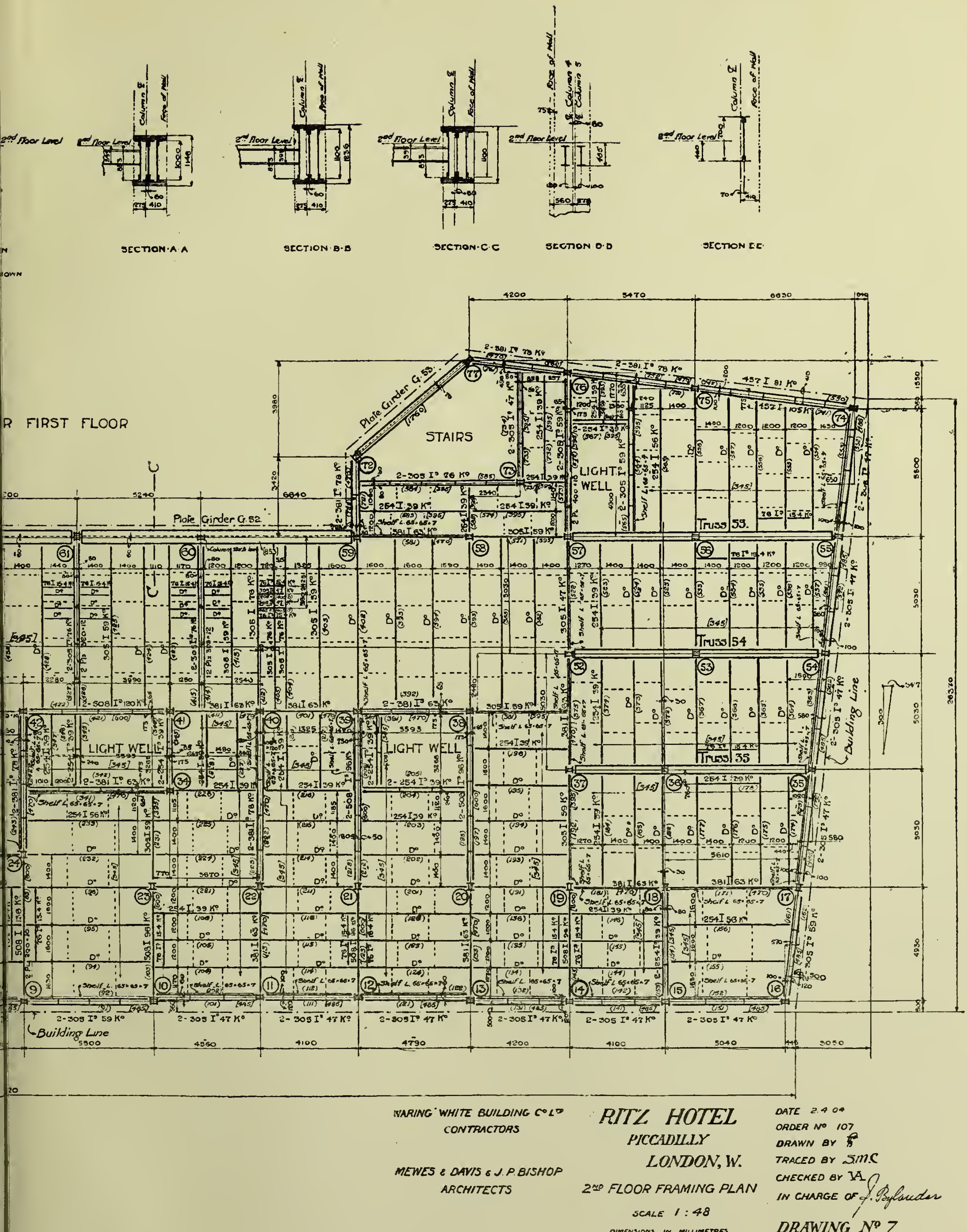


NOTE. Figures in brackets thus [ ] denote the distance from finished floor line to bottom flange of beams.

Figures in brackets thus ( ) denote the number of beams.

Figures in circles thus ◯ denote the number of columns.

For framing round flues see drawing No 27.



G. 160.

[To face page 134.

Brickwork in cement mortar	. 120 lbs.	per cubic foot.	
Granite	. 170	,,	,,
Stone	. 150	,,	,,
Floors, inclusive of steel	. 100	,,	,,

Having ascertained the weights of the walls, the girders carrying them may be selected or designed as described in Chapters V. and VII. The girders carrying the floors may be similarly designed when the weight and loading of the floors is known. These girders, being supported at the surface of the stanchions, put upon them eccentric loads in addition to the concentric load produced by the loads in the upper floors. The necessary section of stanchion may then be calculated as explained in Chapter IX.

Live Loads. — Dead loads upon girders and stanchions are easily, though perhaps tediously, ascertained when the weights of the respective materials are known, and all joists, girders, stanchions, and foundations must be proportioned to carry their share of the loads. Besides the dead load produced by the weight of the building there is a live load to be provided for, consisting of people, furniture, and temporary or movable fittings. The amount of this live load is largely a matter of conjecture. In every case the load allowed for should be, not the average load that will probably come upon the floors, but the maximum load that can reasonably be expected to be possible. In the case of a warehouse the amount of floor load may generally be arrived at easily. Thus if the warehouse be intended for the store of grain, the maximum quantity that can be put upon the floors may be estimated.

In the case of office or hotel buildings the greatest load that could be put upon the floors is so far in excess of the greatest probable load, that it is impossible to arrive at any precise conclusion. It is necessary therefore to be content with allowing for the greatest probable load, leaving it to the factor of safety, used in designing the whole structure, to provide against extraordinary loads. One cwt. per square foot for ordinary office and dwelling rooms, and $1\frac{1}{2}$ cwt. for public rooms or halls, etc., is a satisfactory allowance, and fairly represents average English practice. American practice favours the adoption of considerably lighter loads. $2\frac{1}{2}$ cwts. per square foot is a fair average for warehouse floors, while where heavy machinery is to be expected this may have to be raised to about 4 cwts. The above allowances are supposed to cover the effect of vibration, and may be simply added to the weight of the floor.

It may be considered as practically certain that the total live load, as allowed for above, will never come upon all the floors, and throughout the whole area of all these floors simultaneously, so that it would be obviously extravagant to allow for this possibility, and to proportion columns and foundations accordingly. In fact, the possibility of the maximum load ever being met with diminishes as we pass from joist to girder, girder to stanchion, and stanchion to foundation; besides which the vibration caused by the live load is spread out and taken up throughout the structure, so that its effect will diminish as it passes down the building. It is therefore customary to allow for the maximum live load upon floor joists, and to allow for a varying percentage of this live load upon girders and stanchions. Practice differs considerably as to these percentages, which should vary according to the type of the building. In an average case the several members may be designed for the following loads:—

Joists .	whole dead load	+	whole live load
Girders .	,, ,,	+	85 per cent. live load.
Stanchions	,, ,,	+	75 ,, ,,

In the case of the Ritz Hotel the following loads were allowed for:—

Joists and girders—	whole dead load	+	whole live load.
Stanchions .	,, ,,	+	80 per cent. live load.

It is evident that whatever live load comes upon a girder, this full amount must also be carried by the stanchion which supports it; but it is highly improbable that either the floor below or above will, at the same time, be loaded to a like extent. Thus were a stanchion to support the load of one floor only, the same percentage of the full live load should be allowed upon it as was assumed for the girders; but as the number of floors carried by a stanchion increase, the possibility of the full live load ever coming upon it correspondingly decreases. Thus it might be maintained that a sliding scale should be employed, allowing for a greater percentage of the live load upon the upper part of a stanchion than upon the lower part; and the New York building law allows for the principle, specifying that the total assumed live load shall be allowed upon all floor girders, as well as such part of the stanchions as carry only the roof and top floor. Below this point the percentage of the live load may be decreased by 5 per cent. at each floor, until 50 per cent. is reached, and below this point 50 per cent. is allowed throughout.

The percentages of live load should be greater in the case of warehouses, for here there is a far greater possibility of the maximum load being placed upon a number of the floors, and it will even be well to allow for the full live load throughout.

The same percentage of live load may be considered as acting upon the foundation as acts upon the lower length of the stanchions. (See Chapter XVI.)

Joists and Girders. — Suppose a floor weighing 58 lbs. per square foot is to be carried by rolled steel joists spaced 4 feet apart, which are in turn to be supported by girders 20 feet long and 15 feet apart. Assuming the live load to be 112 lbs. per square foot, then total load $= 58 + 112 = 170$ lbs. per square foot. The amount of floor carried by one joint $= 4 \times 15 = 60$ square feet, and the total load on one joist $= 60 \times 170$

$=10{,}200$ lbs. $=4.55$ tons. By turning to the table in Chapter V. a suitable weight of joist may now be selected. Thus it is seen that a joist 8×4 inches $\times 18$ lbs. would be strong enough to carry the load; but to avoid excessive deflection it would be well to use a size larger, namely, 9×4 inches $\times 21$ lbs.

Assuming that the joists come upon the girder on either side of it, the loading will be 4.55 tons at each point where the joists are supported; or, if only 85 per cent. of live load be allowed for, the loads at these points will be $60\left(58+\frac{85 \times 112}{100}\right)=9180$ lbs. $=4.1$ tons. Referring to Fig. 161, the maximum $BM = 8.2 \times 8 - 4.1 \times 4 = 49.2$ foot-tons $= 49.2 \times 12 = 590.4$ inch-tons.

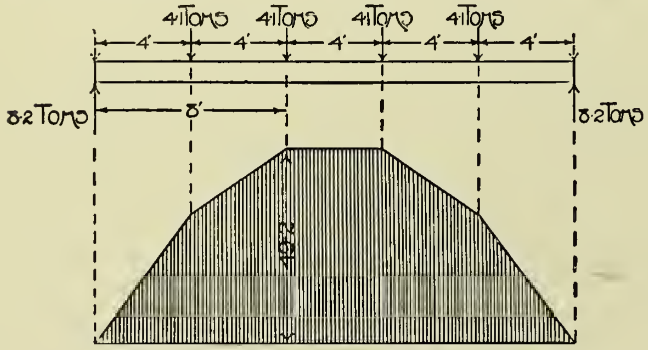


FIG. 161.

$\therefore$ The moment of resistance of the required girder $=\frac{I}{y}f = 590$; and if $f=7.5$ tons per square inch, $\frac{I}{y}=\frac{590}{7.5}=78.7$. $\frac{I}{y}$ is given in the table (Chapter V.) under heading "Resistance about $x-x$," and by looking down this column it is found that a section 15×6 inches $\times 59$ lbs. will be suitable; or two joists 10×6 inches $\times 42$ lbs. may be used, if it is desired to save depth, being placed side by side with separators between.

It should be noticed that the BM will generally be least if the joists that it supports are of even numbers; that is to say, when there is no load at the centre of the girder.

The girders may consist, as above, of single or double rolled steel joists, of plate or lattice girders, or of two rolled steel joists side by side with plates riveted to the top and bottom. The latter BOXED form, although requiring less depth, is to be avoided as far as possible. It is often necessary to keep floor girders of minimum depth, so that the double and box form is largely used. This necessity does not exist in the case of the wall girders. Here, on the other hand, a wide bearing surface is required for the support of the walls, so that the double form is again largely adopted (see Plate VI.), where it will also be noticed that the box form of floor girder has also been used, while wall girders of the plate type may be seen to the left of the photograph. The method of supporting the masonry scaffolding should also be noticed in the photograph.

JOINTS BETWEEN JOISTS AND GIRDERS are frequently made as shown in Fig. 162, the end of one being carefully cut out or "coped" to exactly fit upon the flanges of the other. There is not much advantage in this refinement, and it is decidedly cheaper to cope the joist so as to leave, say, $\frac{1}{4}$-inch clearance as in Fig. 163, trusting entirely to the angle brackets to transmit the

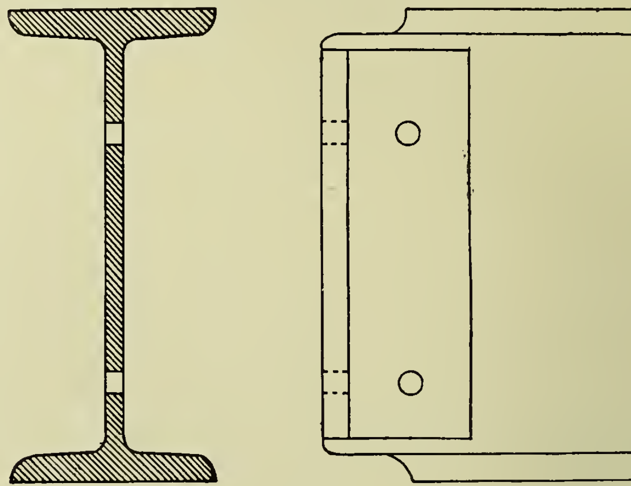


FIG. 162.

load. The various firms supply "standard" angle brackets calculated for the maximum load likely to be put upon a given depth of joist. Where possible it will be best and cheapest to use these standard connections, although it should not be done without satisfying oneself as to their sufficiency. As many rivets must be employed in the angle as will have a

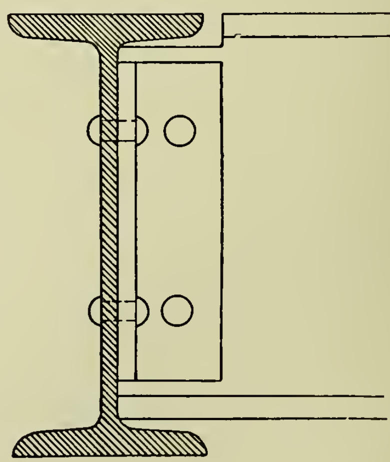


FIG. 163.

total shear resistance equal to the load to be transmitted. In the case considered above the load transmitted $=\frac{4.1}{2}=2$ tons, and using $\frac{3}{4}$-inch rivets with single shear of 2.2 tons one rivet would be sufficient in each arm of the angles; but the connection would be made of the standard pattern as shown in Fig. 163, which has the advantage of increased rigidity.

The method of supporting joists shown in Fig. 164 is particularly useful where the latter are placed moderately close together, an angle being riveted to the web of the main girder throughout its entire length.

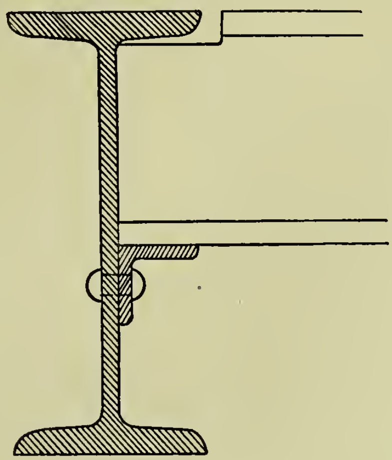


Fig. 164.

Arrangement of Stanchions.—The spacing of the stanchions depends chiefly upon the plan of the building. The spacing of those in the external walls will be governed by the position of window openings, etc., while within the building they should be arranged as far as possible to come at the junction of cross walls, as this will avoid the inconvenience of stanchions standing free in a room or projecting from the walls, besides which the weight of cross walls will be carried immediately to the stanchions without producing concentrated loads upon the cross girders. Excessively large spans should be avoided, but any spacing from 10 to 25 feet will be fairly economical. In the Ritz Hotel the stanchion spacing varies from $13\frac{1}{2}$ to 25 feet. Plate girders 33 to $36\frac{1}{2}$ feet span have been used to carry the back wall, while the spans of the three trussed girders (Fig. 160) vary from $33\frac{1}{2}$ to $37\frac{1}{2}$ feet. The spacing should be as regular as the shape of the building will permit, as by this means a large number of girders and joists will be required of the same dimensions, thereby reducing the cost of the structure and facilitating construction and erection.

It was stated above that the stanchions should come at the junction of cross walls ; but this should not be done to the extent of sacrificing unnecessarily their regular spacing. In planning the building the architect should have this consideration constantly in view, as if it be left until his designs are complete the cost of the steelwork may be largely increased.

CHAPTER XV

THE STEEL FRAMEWORK OF BUILDINGS (*continued*)

Wind Bracing.—Fig. 165 diagrammatically represents the section of a steel framed building. Triangulation has not been adopted, and if the joints are not rigid and there is nothing to prevent it the framework may be blown over by the wind, as indicated in dotted lines. In the tall buildings of America the framework is nearly always trussed,—that is to say, cross ties or "wind braces" are employed as illustrated in Fig. 166. These braces obviously prevent openings being made in the partitions in which they are placed, and to obviate this difficulty other forms of stiffening are also used, such as "knee bracing," lattice girders, and "portal" bracing

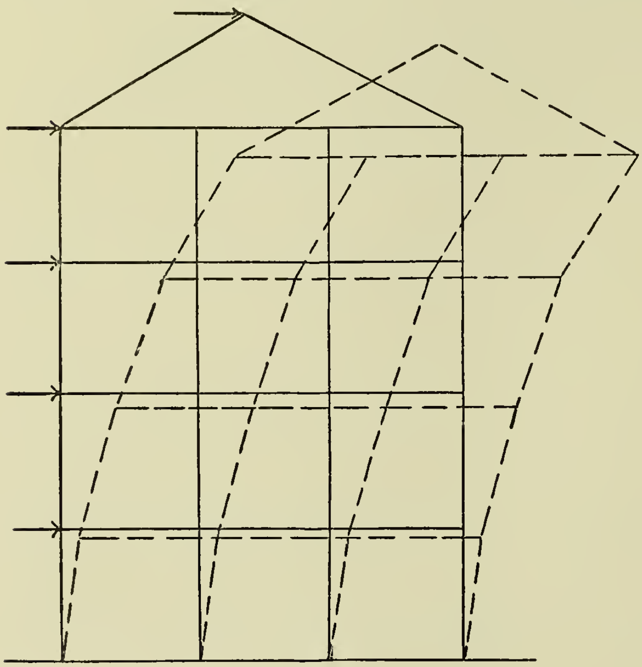


Fig. 165.

(Fig. 167). Where heavy external walls as well as one or two substantial cross walls are employed these obviously take the place of bracing, and prevent deformation. By making some provision against wind pressure thick internal walls may be avoided, and the structure will be rigid while building is taking place, and before all the walls have been filled in.

A wind pressure of 30 lbs. per square foot will be an ample allowance in designing wind bracing, as, although greater pressure may take place, yet the natural rigidity of the structure, together with the weight of the walls, will easily meet this.

Fig. 168 represents the central braced bay in the section of a structure similar to that shown on the left foot side of Fig. 166, with its stress diagram under an assumed wind pressure, being similar to the case of

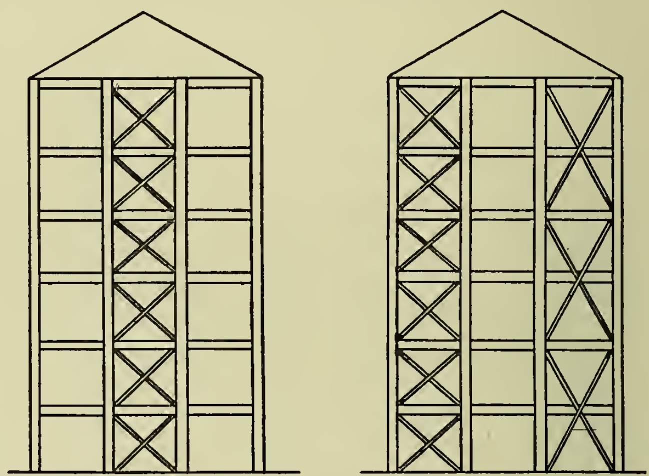


Fig. 166.

the cantilever (Fig. 123). It is seen that the stanchion IF is in compression to the extent of *if*, due to wind pressure alone, and in designing the stanchion this additional load must be allowed for. (*Note.*—To find

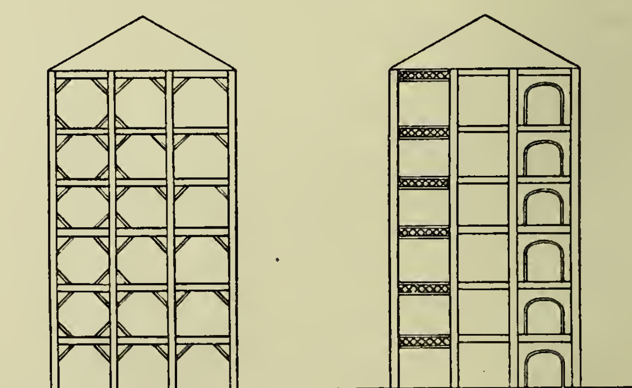


Fig. 167.

stress in right-hand stanchion only, the principle of moments should be used.) Where the place of the diagonal ties is taken by knee braces, by rigid joints, or by solid wall the compressive stress in the right-hand stanchion would still be as found above. If all joints of the structure were braced, as shown in Fig. 167 and on the right of Fig. 166, the effective base

would become much widened, and consequently the compressive stress would not only be considerably diminished, but would be divided between the two right-hand members.

So far as Fig. 168 is concerned, the probability is that the cross braces would all be designed as ties

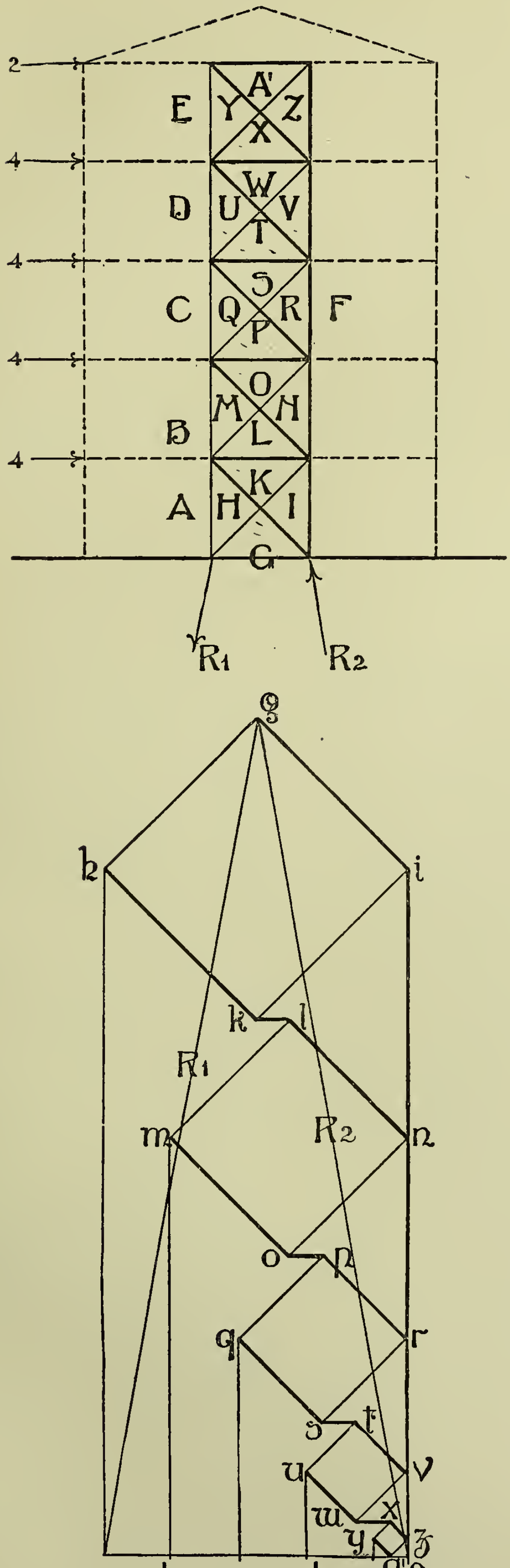


Fig. 168.

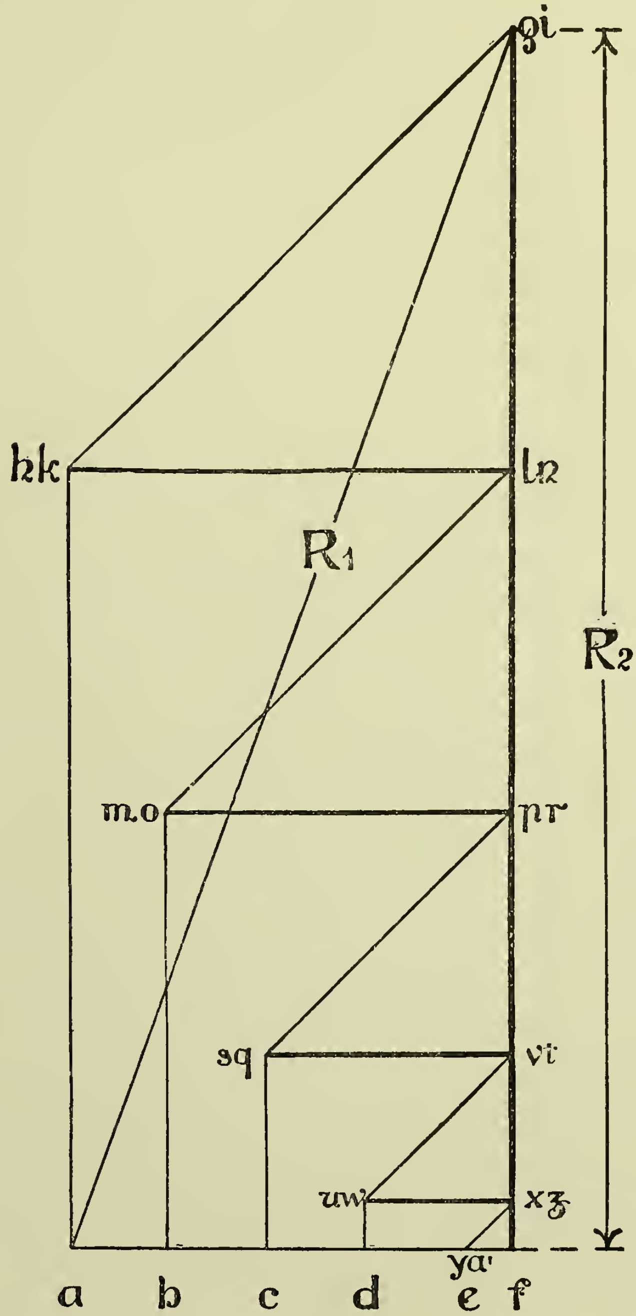


Fig. 169.

only. In such a case their compressive resistance must be neglected, and the stress diagram would be as shown in Fig. 169.

Rigid Joints.—In buildings of such a height as is allowed in London it will be unnecessary to carefully consider wind loads unless the width of the building is

very small, while the height is three or more times its width. Although it is here generally unnecessary to consider wind pressure, yet all joints and connections should be made thoroughly rigid. In the erection of a tall building, where the walls are to be relied upon to give stiffness to the structure, temporary struts of

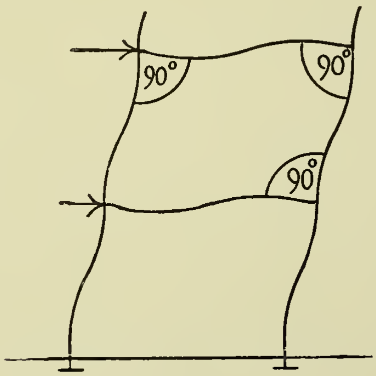


Fig. 170.

timber may be fixed across the vertical bays to meet any tendency to sway before the walls are built.

Fig. 170 indicates the bending tendency upon girders and stanchions when wind pressure is met by the general rigidity of the framing without the aid of bracing.

Another effect of rigid joints is illustrated in Fig.

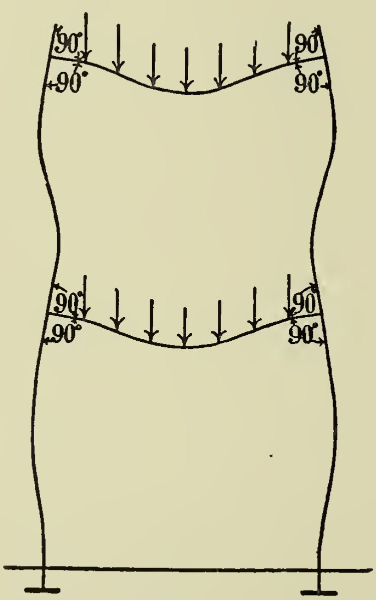


Fig. 171.

171. Assuming that the ends of the cross girders are perfectly and rigidly connected to the stanchions, the girder becomes a fixed beam, with BM at its extremities equal approximately to $\frac{Wl}{12}$ (see Chapter III.). This bending moment must be resisted by the stanchion.

The effects of BM in the cases of the stanchions just considered are very slight: for in the first case substantial walls will assist to resist the wind pressure; while in the second case, in order to thoroughly fix the ends of the girder, many more rivets would be needed than are actually put into the joint, which will, in most cases, be sufficiently flexible to avoid any considerable BM from this cause. Nevertheless these tendencies exist, and, although it will seldom be necessary to calculate their amounts, it will be well to bear them in mind.

We must now decide whether the portions of stanchion between floors may be considered to have fixed or hinged ends. (The reason that a continuous member may be considered to have hinged ends was shown in reference to roof rafters, Fig. 143.)

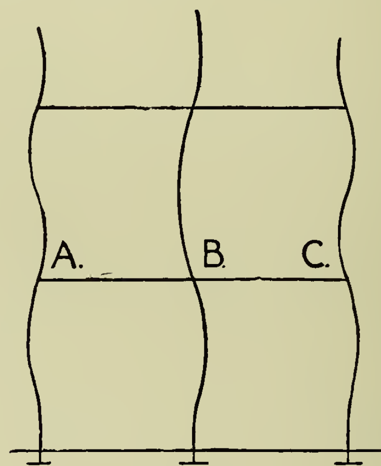


Fig. 172.

It might be maintained that the joints of the girders were sufficient to "fix" the ends of each portion of the stanchion; but the effect of considering the girders to have "fixed" ends, which would have called for more metal in the stanchions, has already been laid aside, so that it would clearly be inconsistent now to consider the same joint as fixing the ends of the stanchions. Therefore where the load upon a stanchion is concentric, or the loadings on either side of it balance one another, the portions of stanchion between joints may be considered to have hinged ends, as at B, Fig. 172; but where the stanchion being eccentrically loaded is compelled to deflect as indicated at A and C the portions of stanchion may be considered to have fixed ends, but in this case additional metal is required to meet the eccentric load, as explained at end of Chapter IX.; but in any case there should be sufficient metal to resist the load as a concentric load, considering the ends as hinged.

The above conditions will only exist so long as the live loading on every floor is uniform. Thus in Fig. 172, if the loading on the left half of the floor be greater than that on the right, an eccentric load will be put upon the central column. Again, if one floor be fully

loaded while the one below is not, the curve of the outer stanchions A and C will incline more nearly to the form of the stanchion with hinged ends. The latter effects are compensated by the fact that, if these conditions exist, the structure will not, at the same time, be loaded to its full extent.

Supposing it necessary to find the section for a portion of a stanchion 14 feet long, receiving a concentric load of 80 tons due to floors above, 10 tons from each of two girders supporting the face wall and coming upon the stanchion on opposite sides of it, and 10 tons from a girder supporting the floor (see Fig. 173). The stanchion has thus a total load of 110 tons, of which 10 tons is an eccentric load. A section 12 x 12 inches may be selected tentatively for this. According to the table

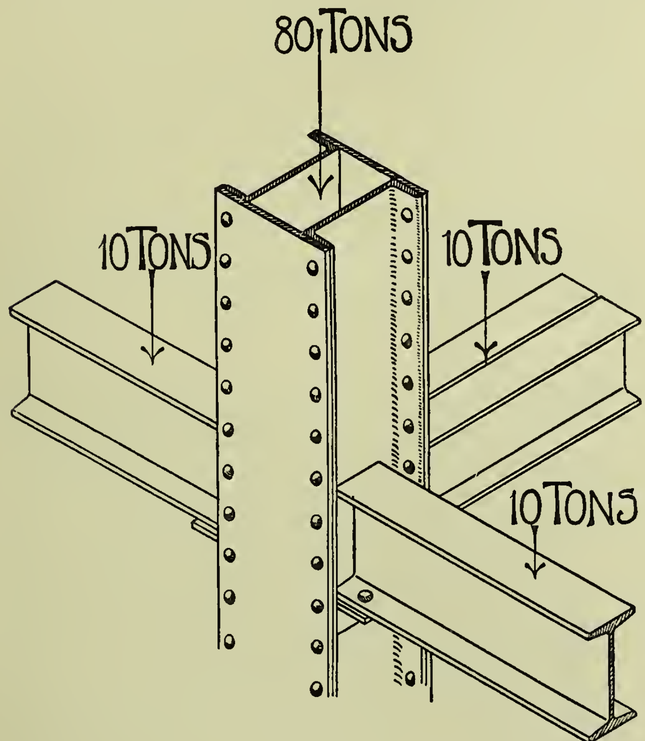


Fig. 173.

in Chapter VIII., the approximate value of $r = .3\text{D} = 3.6$ inches; whence $\frac{l}{r} = \frac{168}{3.6} = 46.6$; and from the table at the end of Chapter VIII. the safe load = 5 tons per square inch, requiring 22 square inches sectional area to resist the total load applied concentrically. Assuming that the amount of eccentricity of the point of application of floor girder = 8 inches, and putting $f = 6.5$; then $\text{A} = \frac{\text{W}dy}{r^2 f} = \frac{10 \times 8 \times 6}{3.6^2 \times 6.5} =$ say $5\frac{1}{2}$ inches (see end of Chapter IX.).

∴ Total area necessary = approximately $22 + 5\frac{1}{2} = 27\frac{1}{2}$ square inches. Two channels $10 \times 3 \times \frac{7}{16}$ inches and 2 plates $12 \times \frac{1}{2}$ inch gives an area of $25\frac{1}{4}$ square inches (see Fig. 174). This section may be tested by more accurate methods.

About axis XX, $\text{I} = 2\text{I}_1 + \frac{\text{B}}{12}(\text{D}^3 - d^3)$. (See Chapter II.)

The moment of inertia of the channel section as found from manufacturers' tables $= \text{I}_1 = 87.75$.

$$\therefore \text{I} = 2 \times 87.75 + \tfrac{12}{12}(11^3 - 10^3) = 506.5.$$

$$r_x^2 = \frac{\text{I}}{\text{A}} = \frac{506.5}{25.25} = 20.06.$$

$$\therefore r_x = 4.48.$$

About axis YY, $\text{I} = 2\left(\frac{m\text{B}^3}{12} - \frac{nb^3}{12}\right) + \frac{d}{12}(b^3 - p^3)$.

$$= 337.$$

$$\therefore r_y^2 = \frac{337}{25.25} = 13.35.$$

$$\therefore r_y = 3.65.$$

Considering the stanchion as hinged at its ends, and taking the lesser value of r, then $\frac{l}{r_y} = \frac{168}{3.65} = 46$;

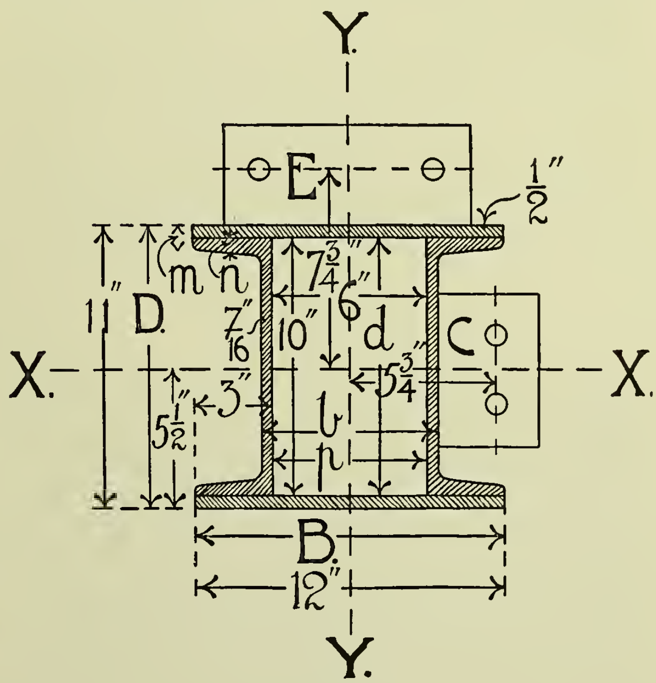


Fig. 174.

whence, from the table at the end of Chapter VIII. the safe load = 4.5 tons per square inch; and the necessary area = 24.2 square inches.

Now, assuming the floor girder to be attached to an angle bracket between the flanges as at C, Fig. 174, the eccentricity $= 5\frac{3}{4}$ inches; and considering the ends of stanchion as fixed, as before $\frac{l}{r_y} = 46$, and safe load = 5.02. Hence the necessary area for concentric loading only . . . . $= \frac{110}{5.2} = 22$ sq. ins.

$$\text{A} = \frac{\text{W}dy}{r^2 f} = \frac{10 \times 5\frac{3}{4} \times 6}{13.35 \times 6.5} = \text{approximately} \quad 4 \quad ,,$$

$$\therefore \text{Total area necessary} = 26 \quad ,,$$

Now, considering the floor girder as attached to

the flange, as at E, Fig. 174, $r = 4.48$ inches, while eccentricity $= 7\frac{3}{4}$ inches.

$\therefore \frac{l}{r_x} = \frac{168}{4.48} = 37.5$; and for fixed ends the safe load $= 5.12$ tons per square inch. Hence the necessary area for concentric loading only $= \frac{110}{5.12} = 21.5$ sq. ins.

$$A = \frac{Wdy}{r^2 f} = \frac{10 \times 7\frac{3}{4} \times 5.5}{20.06 \times 6.5} = 3.25 \quad ,,$$

$$\therefore \text{Total area necessary} = 24.75 \quad ,,$$

It is seen that, although eccentricity is less when the floor girder is attached between the flanges than when attached to the plates, yet the resistance to bending is also less in this direction, and in the example we have been considering the stanchion was found to be stronger when the floor girder was attached to the flanges, as in Fig. 173.

When metal has to be added to a section to meet a large eccentric load it may be economically done by apportioning it to the sides which are at right angles to the girder which conveys the load; but care must be taken to see that the stanchion is sufficiently strong in the other direction also.

Joints, Girders, and Stanchions.—Plate VII. contains four photographs of joints between girders and stanchions in the Ritz Hotel. It will be seen that these have been made with angle brackets attached to both top and bottom flanges in order to obtain a rigid connection, while the lower brackets are stiffened by further angles placed vertically. The number of rivets through a stanchion, by means of which a girder is fixed to it, must give a resistance to shear at least equal to the load conveyed by the girder; but it will be seen in the illustrations that a considerably larger number have been used than would then be called for. Not only is greater rigidity secured by this means, but the stanchion is materially assisted in taking up and spreading over its section the load brought on it by the girder. The third photograph on the plate shows the wall girders, consisting of two rolled steel joists, and a floor girder

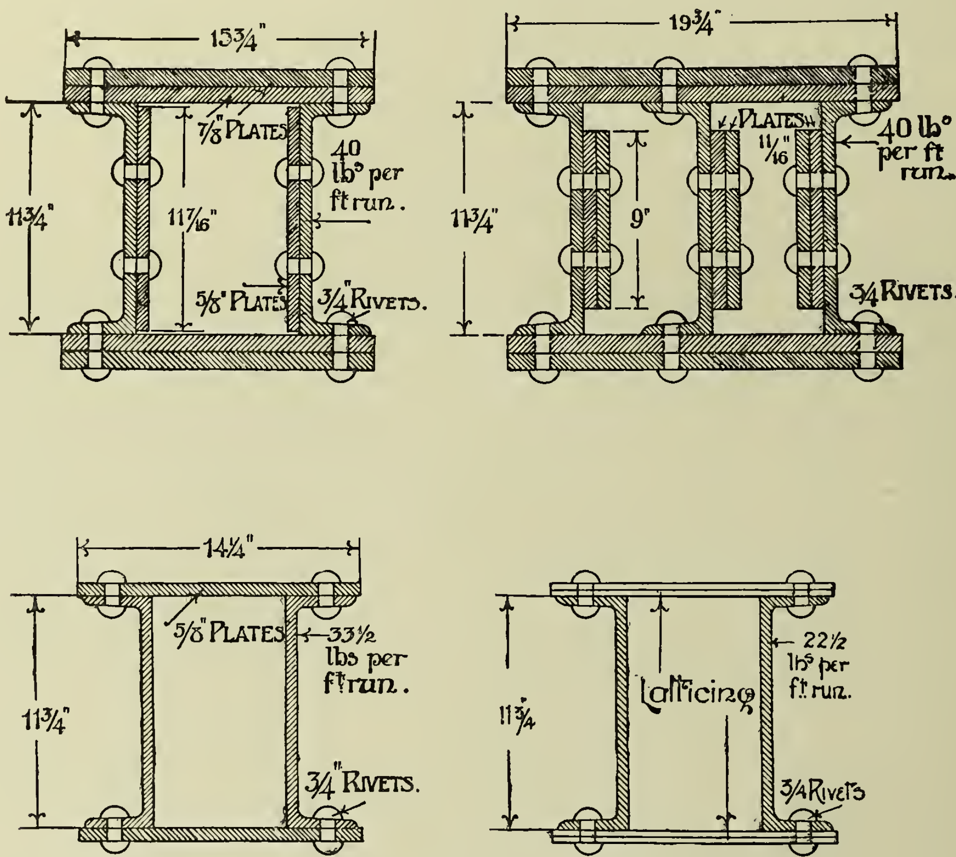
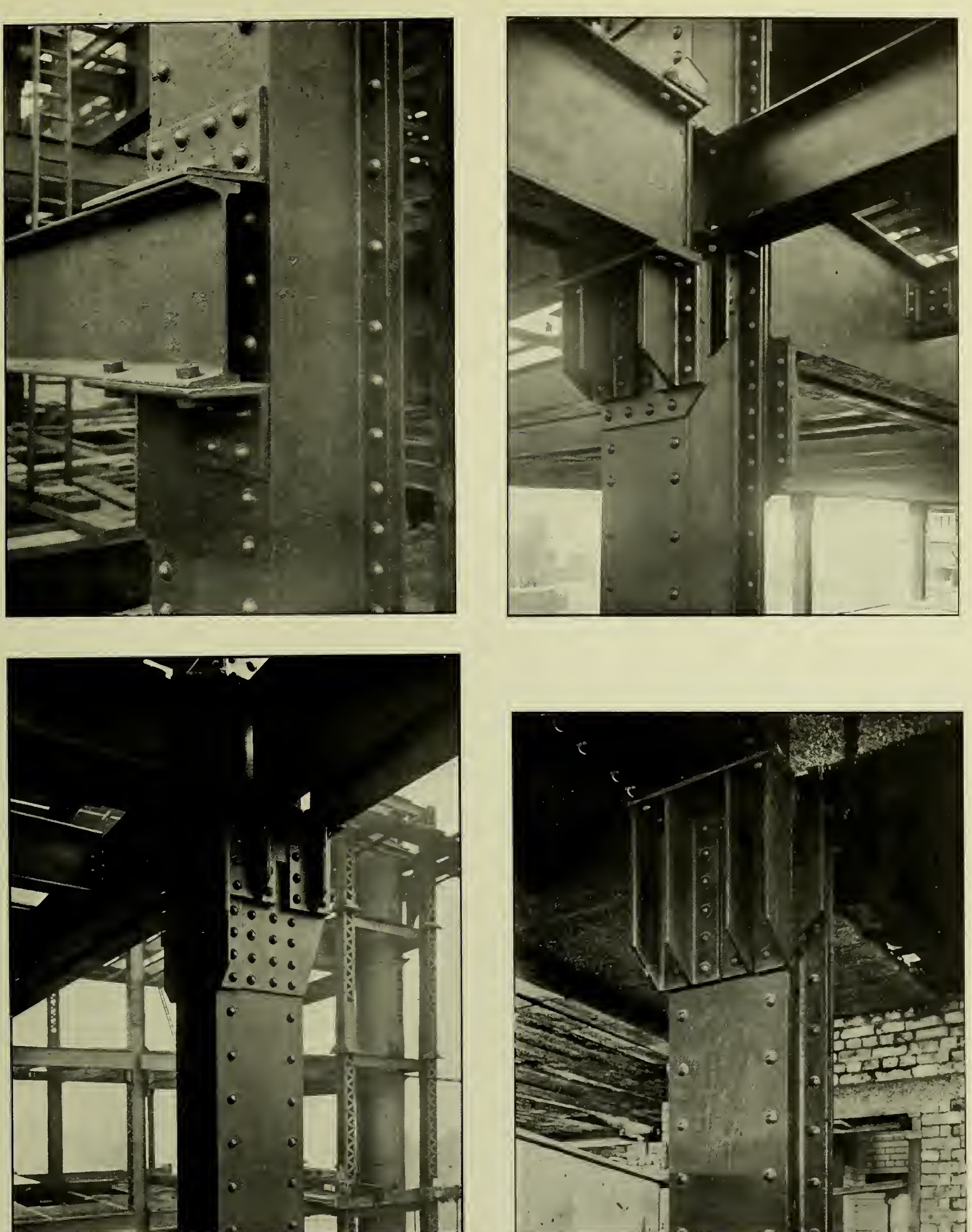


Fig. 175.

of box form; while the connection of a floor joist, which in this case is made to the wall girder, may also be seen.

It will be noticed that the wall girders are not placed opposite to the centre of the stanchion, but are fixed nearer to the outside of the building, as may also be seen in Plate VI. The effect of this eccentric loading is to produce a bending moment in the stanchion opposite to that produced by the floor girder. Thus, by carefully arranging floor and wall girders, the BM's due to dead loads may be balanced so as to entirely eliminate the bending effect upon the stanchion. Live load, however, will still produce a bending moment.



RITZ HOTEL, PICCADILLY, LONDON.
Joints between Girders and Stanchions.

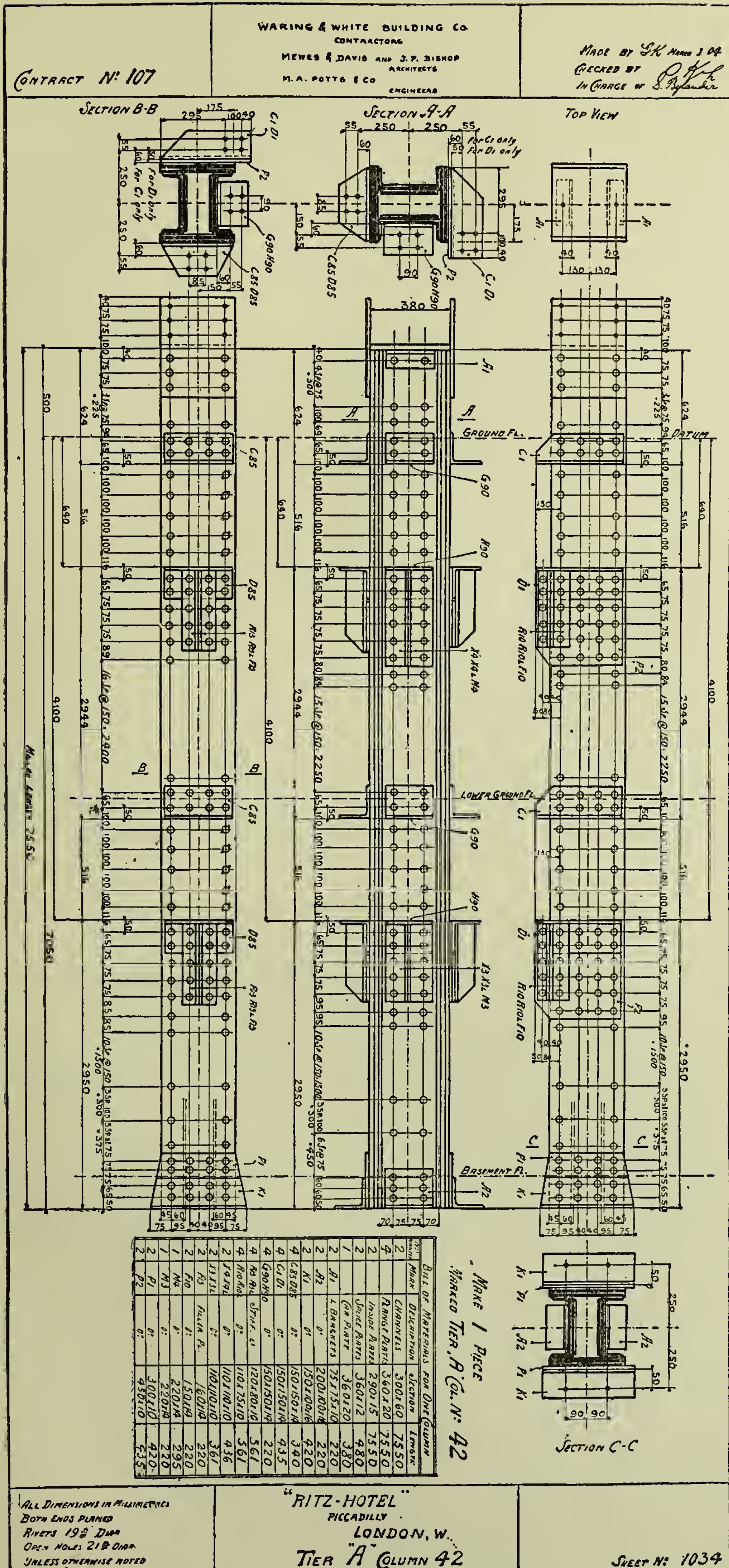


Fig. 176.

Section of Stanchion.—The forms of stanchions were considered in Chapter IX., and any of the sections shown in Fig. 107 may be used in skeleton construction; zed-bars have been largely used in America for forming the stanchions, and cast-iron stanchions have also been used to some extent. As before noticed, cast iron is more reliable than steel when improperly protected against fire and corrosion, but with proper protection steel is undoubtedly the more reliable, and its use for the purposes now being considered has become practically universal.

In order to facilitate connection between lengths of stanchions they should preferably be of a form whose sectional area may be changed in different portions of its height without materially altering the design or the outside dimensions.

Fig. 175 shows the sections of some of the stanchions used in the Ritz Hotel, the measurements being here shown converted intoEnglish measure. It will be noticed that one outside dimension has been kept practically constant throughout. Fig. 176 shows the working detail of one of the stanchions. The detail does not show the true length of the stanchion, for it is broken wherever the riveting is uniform and where there are no connections. It will be seen that all dimensions and the varying spacing of rivets are clearly figured.

The Phœnix form of stanchion (Fig. 107, "J") is thoroughly reliable and economical of metal; but, being of a special section, unless a large quantity of the same section be required the price per ton may be high, and there may be considerable delay in delivery. The difficulty of forming joints and connections also militates against its use.

The zed-bar section has the objection that zeds are not commonly rolled in England, although they are fairly common in America. This form has the advantage that all surfaces are exposed and can be thoroughly painted and protected.

The use of the lattice form may be seen in the third photograph on Plate VII., while Plate VIII. shows its

application to the roof construction. This form is excellently adapted for light loads which are applied concentrically. However, if the load be applied eccentrically to the side of the stanchion (as in Fig. 177), the load is carried almost entirely by that half of the stanchion to which it is directly fixed, for the latticing will assist very little in distributing the pressure throughout the section. The same objection, to a lesser degree, applies also to the zed-bar section.

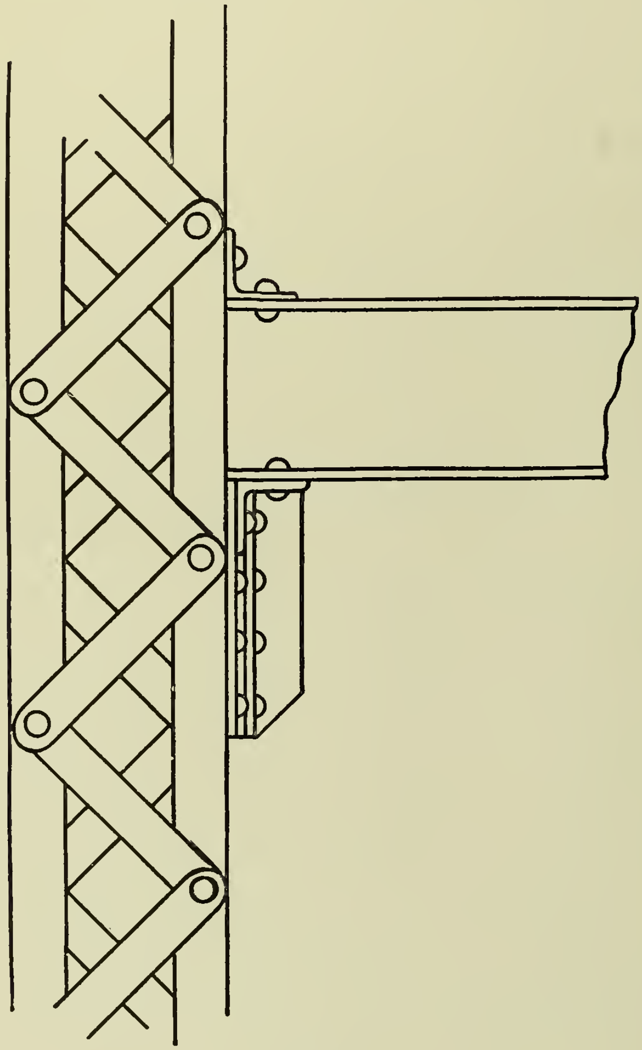


Fig. 177.

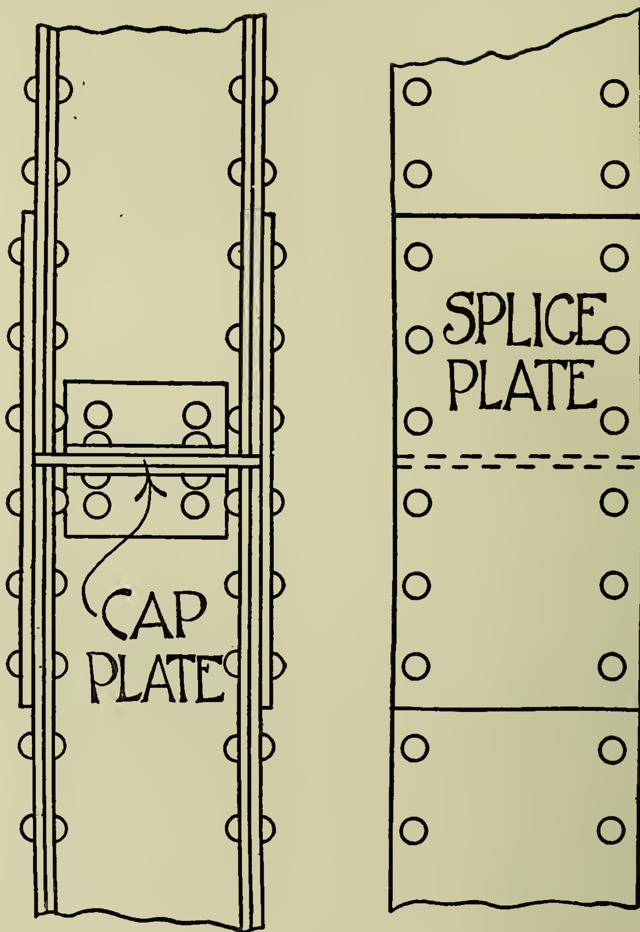


Fig. 178.

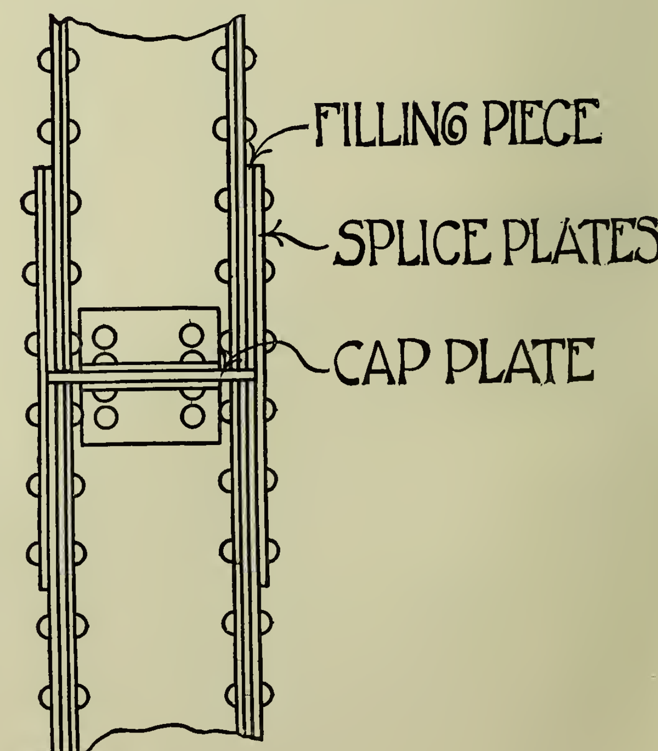
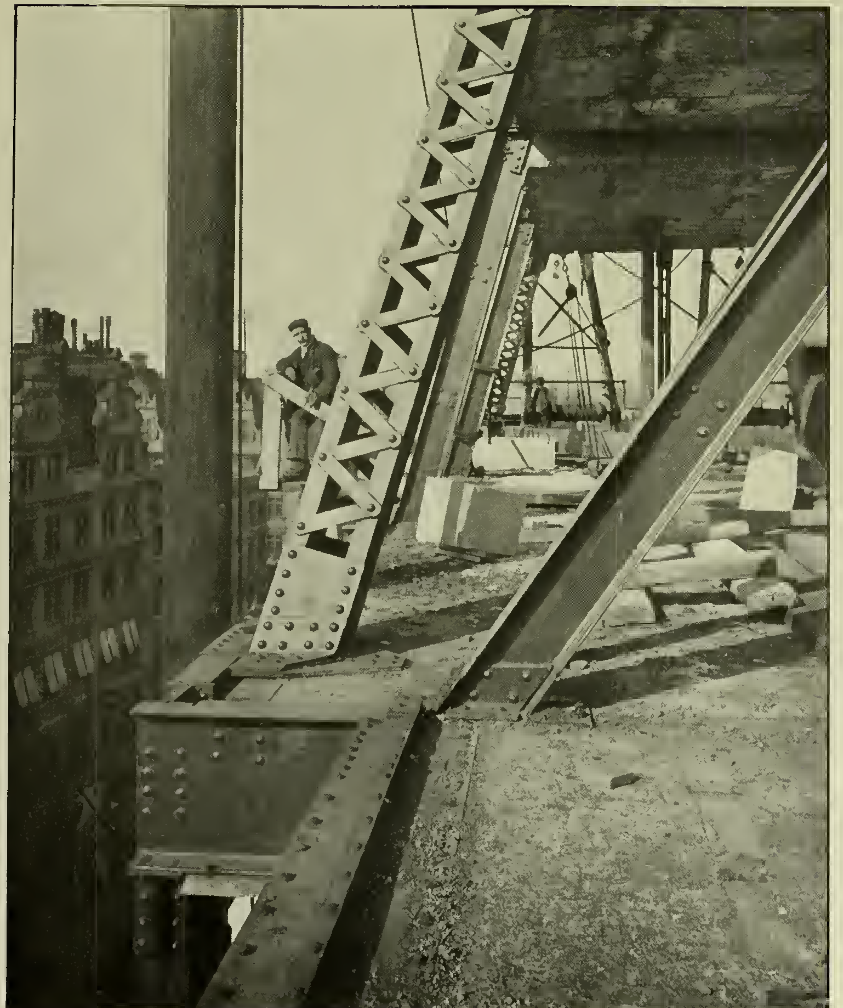


Fig. 179.

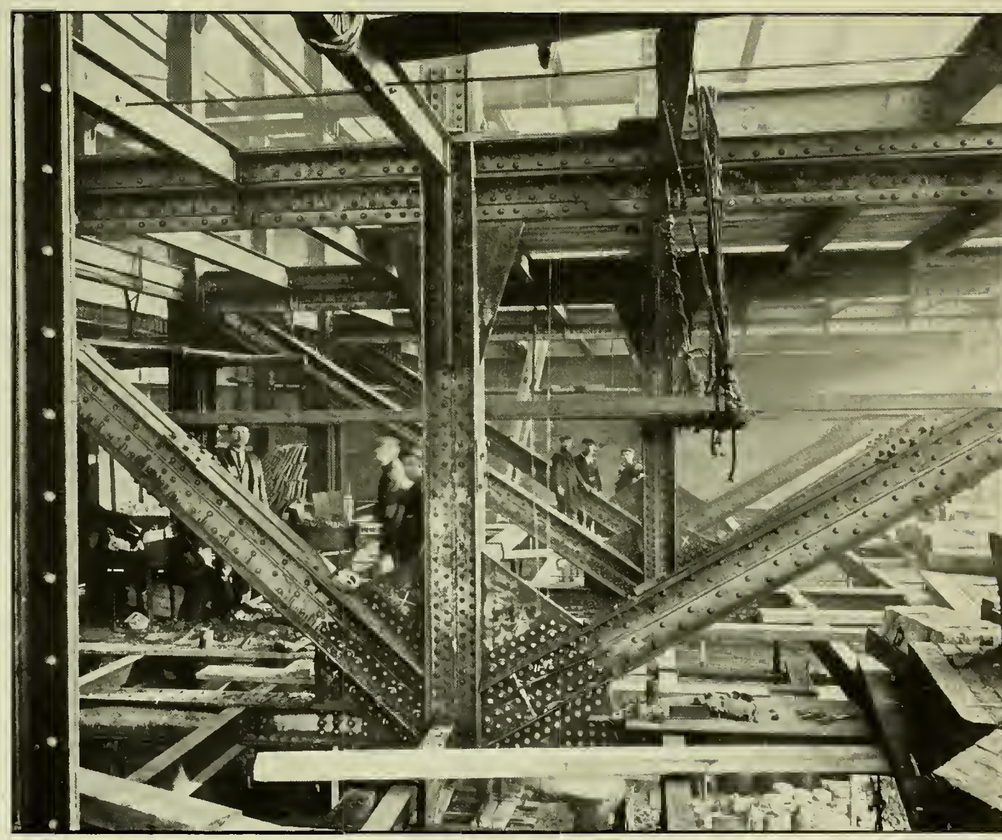
Stanchions are generally made in two and sometimes in three-storey lengths. The advantage of this practice over that of employing separate lengths for each storey is obvious. It is sometimes the practice to make the bottom lengths of the stanchions alternately of one and two-storey lengths, so that adjacent stanchions will break-joint with one another. It will generally be best to make the whole of a single length of the same section throughout; but if there is a large increase of load on the lower half, the section may be altered by adding plates to this part.

Stanchion Joints.—The joints between lengths of

PLATE VIII.



Lattice Stanchion in Roof.



Joint of Floor Trusses.

RITZ HOTEL, PICCADILLY, LONDON.

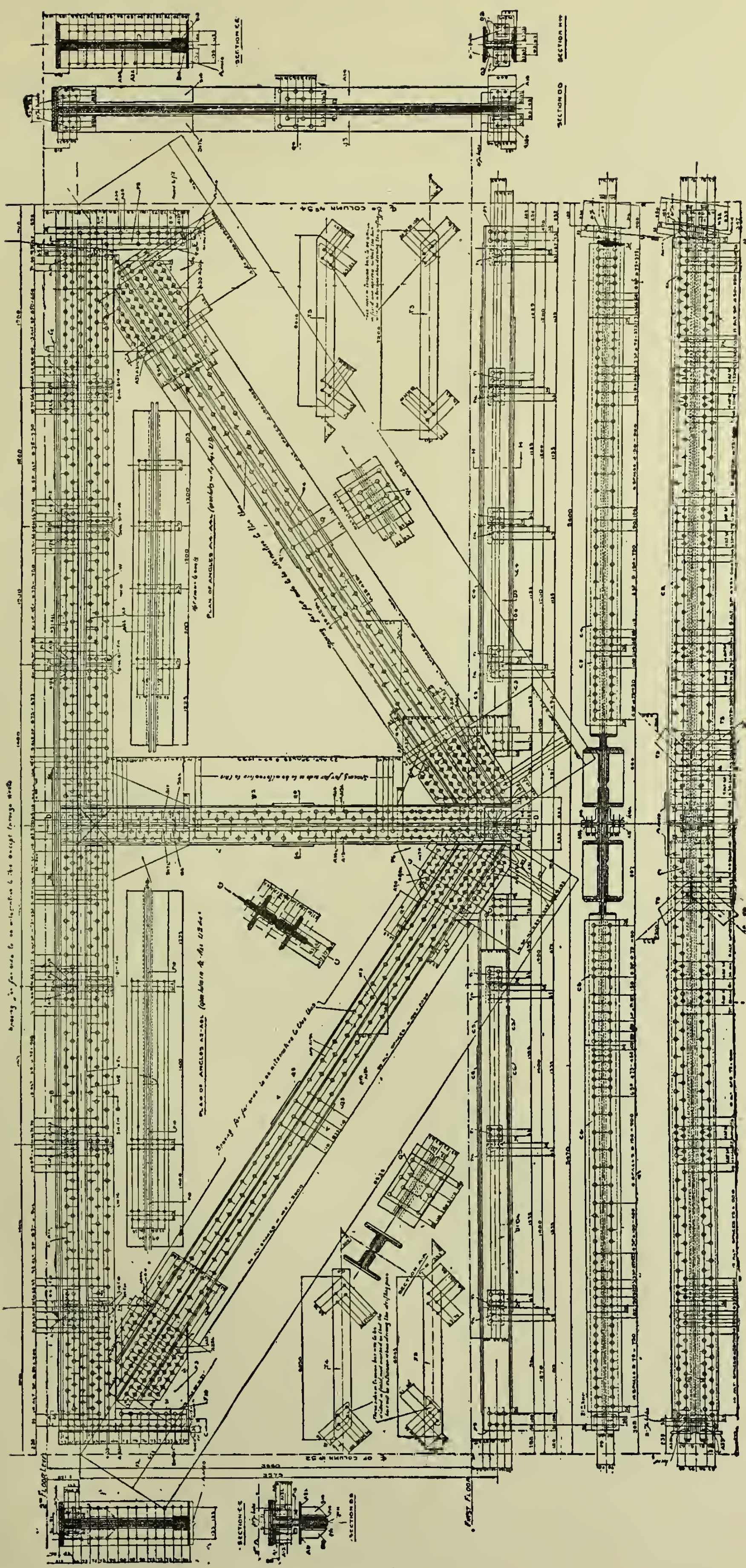


Fig 180.

stanchions are usually made slightly above a floor level, so as to avoid interfering with girder connections, and in order that the girders may be fixed before another length of stanchion is added. In the Ritz Hotel these joints are made 18 inches above floor level (see Plate VI. and the first photograph on Plate X.).

To form the joints the ends of the stanchion are "faced" accurately, and a butt joint is made with "splice plates" on either side (Fig. 178). If the form of the stanchion alters in the two parts to be connected a plate must be inserted between the two ends, as shown; but it will be well to employ these plates in any case. If the outside dimensions of the stanchions alter, filling pieces must be inserted beneath the splice plates, as shown in Fig. 179.

The splice plates are fixed to the top of the lower length of stanchion when it is being constructed in the shops, as may be seen in Fig. 176. The connection to the upper portion must be riveted in the field. The method of denoting in black those rivets which are to be field riveted should be noticed in Fig. 176.

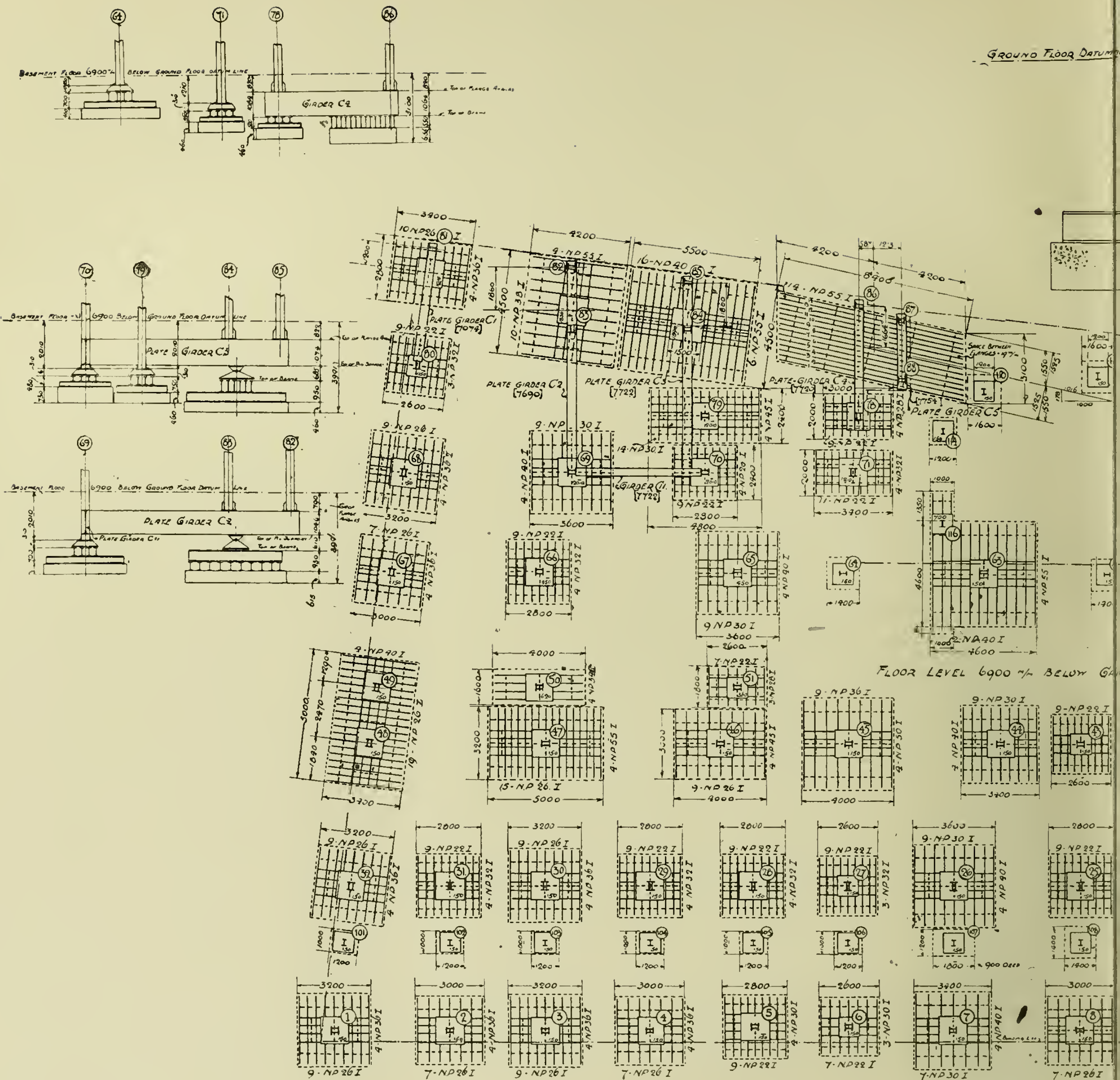
The number of rivets used in a splice is a matter of judgment, their use being simply to make a stiff joint, the compressional stress being directly transmitted at the butt joint.

Bolts should never be used in any important part of the structure, except where unavoidable; but if it is necessary to employ them they should be very carefully fitted.

Fig. 180 is the working drawing for one of three similar trusses used to carry the floor over the restaurant in the position shown on plan, Fig. 160. It was desired to preserve an even ceiling over the restaurant, and this prompted the adoption of this construction. These trusses are of the full depth between two floors, and each one carries a stanchion, at the centre of its span, which extends throughout the upper floors. Apart from the special need for these trusses they add to the general stiffness of the structure. The second photograph on Plate VIII. shows the joints at the centre of the span.

The bill of materials is an item that ought always to be attached to a working drawing before it is sent to the shops to be worked to. By consulting this table it will at once be seen what material is needed and what lengths have been ordered from the mills for this particular detail, thus avoiding the danger of using material intended for other parts of the work; for it is usual to order only just so much material as shall be needed for the work in hand. Such a table may be seen in Fig. 176. There is no need, however, for the architect or engineer to make out this table, it being solely a matter for the contractor.

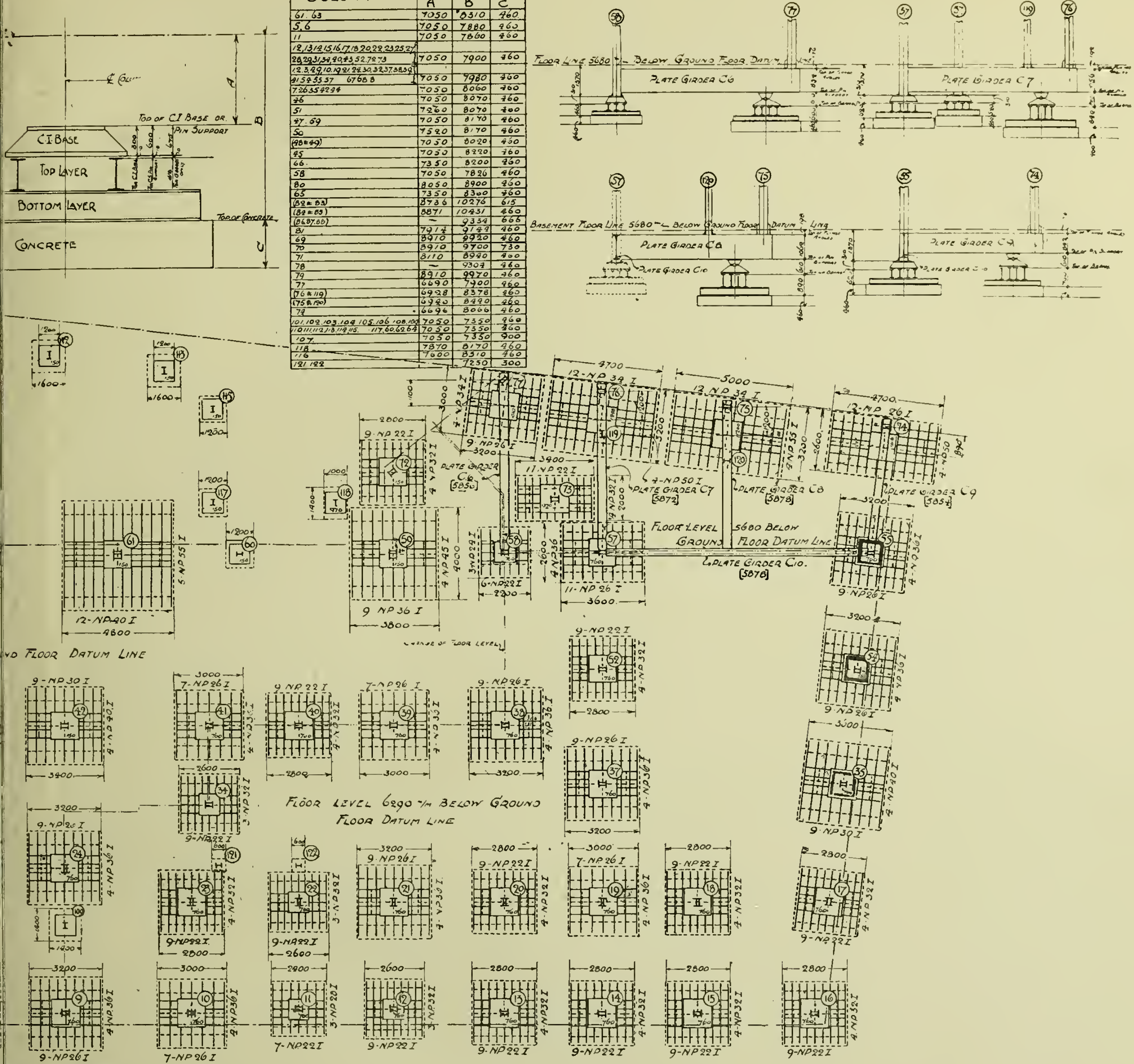
It may be mentioned here that the large number of illustrations in this and the previous chapter of the Ritz Hotel, Piccadilly, are introduced by permission of the engineer, Mr. S. Bylander, and of the proprietors of the *Builders' Journal*.



— · SECTIONS · OF ·

MARK	WEB	FLANGE ANGLES
C1	2 PLATES 800 × 12 1 do 800 × 18	8 ANGLES 150 × 150 × 14
C2	2 PLATES 1000 × 14 1 do 1000 × 20	8 ANGLES 150 × 150 × 14
C3	2 PLATES 1000 × 14 1 do 1000 × 20	8 ANGLES 150 × 150 × 14
C4	2 PLATES 1000 × 16 1 do 1000 × 20	8 ANGLES 150 × 150 × 14
C5	2 PLATES 1000 × 12	4 ANGLES 150 × 150 × 14
C6	2 PLATES 800 × 12 1 do 800 × 18	8 ANGLES 150 × 150 × 14
C7	2 PLATES 1000 × 16 1 do 1000 × 20	8 ANGLES 150 × 150 × 14
C8	2 PLATES 1000 × 16 1 do 1000 × 20	8 ANGLES 150 × 150 × 14
C9	2 PLATES 800 × 14 1 800 × 20	8 ANGLES 150 × 150 × 14
C10	1 PLATE 990 × 14	4 ANGLES 150 × 150 × 14
C11	1 PLATE 990 × 10 2 do 400 × 14	4 ANGLES 150 × 100 × 14

FIG. 181.



COLUMN	DIMENSION AT A	B	C
61. 63	7050	8310	460
5.6	7050	7880	460
11	7050	7860	460
12,13,14,15,16,17,18,20,22,23,25,27, 28,29,31,34,40,43,52,72,73	7050	7900	460
1,2,3,4,9,10,19,21,24,30,32,37,38,39, 41,54,55,57 67,68,8	7050	7980	460
7.26,35,42,44	7050	8060	460
46	7050	8070	460
51	7260	8070	400
47.59	7050	8170	460
50	7520	8170	460
(48 & 49)	7050	8020	460
45	7050	8220	460
66.	7350	8200	460
58	7050	7826	460
80	8050	8900	460
65	7350	8300	460
(82 & 83)	8736	10276	615
(84 & 85)	8871	10431	460
(86,87,88)	—	9334	666
81	7914	9149	460
69	8910	9920	460
70	8910	9700	730
71	8110	8940	400
78	—	9307	460
79	8910	9970	460
77	6690	7900	460
(76 & 119)	6928	8378	460
(75 & 120)	6940	8490	460
74	6696	8066	460
101.102.103.104 105.106 108.109	7050	7350	460
110,111,112,113,114,115. 117,60,62,64	7050	7350	460
107.	7050	7350	900
118	7870	8170	460
116	7600	8310	460
121 122		7250	300

ATE· GIRDERS·—

TOP FLANGE	BOTTOM FLANGE
1 Plate 700×12 1 do 700×12	1 Plate 700×10 1 do 700×10
1 Plate 700×14 1 do 700×16	1 Plate 700×12 1 do 700×14
1 Plate 700×18 1 do 700×22 1 do 700×22	1 Plate 700×18 1 do 700×18 1 do 700×18
1 Plate 700×20 1 do 700×20 1 do 700×20	1 Plate 700×18 1 do 700×18 1 do 700×18
1 Plate 650×12	1 Plate 650×20
1 Plate 700×12 1 Plate 700×12	1 Plate 700×10 1 do 700×10
1 Plate 700×14 1 do. 700×14 1 do. 700×14	1 Plate 700×12 1 do 700×12 1 do. 700×12
1 Plate 700×16 1 do 700×16 1 do. 700×16	1 Plate 700×14 1 do 700×14 1 do. 700×14
1 Plate 700×12 1 700×12	1 Plate 700×10 1 do 700×12
1 Plate 350×16 1 do 350×16 1 do 350×16	1 Plate 350×14 1 do. 350×14 1 do 350×14

NOTE — Figures in brackets thus [ ] are the dimensions in millimetres from the top of top flange angles (of plate girder) to Ground Floor Datum Line.

Small figures shewn in corner of bases are the dimensions in millimetres from top of base to the Basement Floor Level at that point.

When one side only of footing is dimensioned the footing is square.

"RITZ-HOTEL"

LONDON, W.

GRILLAGE . PLAN.

SCALE . 1 : 96

DIMENSIONS IN MILLIMETRES

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CHAPTER XVI

STEELWORK IN FOUNDATIONS

THE question of foundations for ordinary masonry construction was considered in Chapter I. of Part III. Volume I.; but the necessity of carefully proportioning foundation areas to the loads to be borne, in order to ensure equality of settlement, is particularly important in connection with steel frame structures, where large concentrated loads have to be dealt with. It must be borne in mind that it is here not only wasteful to put unnecessary material in any one part,—but it is actually prejudicial to the building.

It is obvious that the foundation of a stanchion must support the same load as does the stanchion itself, and it might be thought that the foundations should be proportioned to the same loading that was assumed in designing the stanchions; but in arriving at this load a large live load has been allowed for. The maximum live load will very rarely take place, while, for several months after the weight of the building comes upon the foundations, the live load will probably be almost *nil*; in fact, the greater part of the settlement which is bound to take place will have occurred before any appreciable live load is applied. It is therefore better to design the foundation in proportion to the dead load only, for otherwise the wall stanchions will settle more than those which carry floor loads only. The effect of the possible live load cannot, however, be entirely neglected, for when this takes place the foundations would then be overloaded. It may therefore be allowed for by taking a smaller safe load per square foot of foundation bed.

The safe load may be arrived at as follows. Suppose one stanchion of a building, representing the average condition of those stanchions which support dead and live loads, is loaded with a dead load of 75 tons and a live load of 25 tons, and that the earth is capable of bearing 2 tons per square foot with safety. The total load to be carried = 100 tons, and the necessary area = $\frac{100}{2} = 50$ square feet. Now, considering dead load only, the load per square foot $= \frac{75}{50} = 1\frac{1}{2}$ ton; and this unit may be taken in calculating for the rest of the foundations.

In the case of a warehouse a heavy live load is probably to be expected as soon as the building is erected, and in this case it will be well to allow for a certain percentage in proportioning the area of the foundations.

The following list gives average values for the safe loads upon various earths.

The values given must be regarded only as approximate, as the bearing capacity of similar earths may vary considerably under slightly different circumstances. For all works of importance practical tests should be conducted.

TABLE OF APPROXIMATE SAFE LOADS UPON EARTHS.

Material.	Safe Load. Tons per Sq. Ft.	Material.	Safe Load. Tons per Sq. Ft.
Rock	5 to 30	Clay, hard, compact, dry . . .	4
Gravel— Compact . .	8	Clay, moderately hard	2 to 3
Loose . . .	2 to 4	,, soft . . .	1
Sand	2 to 5	Rammed earth . .	1
Common earth .	2 to 4	Silt, etc. . . .	$\frac{1}{2}$

THE FORM OF FOUNDATION in most general use, and one which is suitable for all soils of moderate bearing capacity, is known as the grillage foundation, in which steel beams are used to spread the load over a large surface. Where the soil at the natural level of the footings is of a poor nature, and where a solid stratum exists below, which is easily reached, it is well to carry piers down to this stratum. Where the soil is very poor, and where a firm stratum exists within a reasonable depth, piers may be built up from this stratum by means of a well or caisson construction (Chapter I. Part III. Vol. I.). Where again the soil, to a considerable depth, is of bad or uneven quality, so that equal settlement cannot otherwise be obtained, timber piles may be used in conjunction with the grillage form of foundation; but if permanency is to be expected the timber should rest entirely below the water level, in order to avoid the evil effects of varying conditions. Concrete piles are more desirable than timber ones (see Vol. V.).

GRILLAGE FOUNDATIONS.—Fig. 181 shows the plan of foundations, or grillage plan, of the Ritz Hotel, in which each stanchion is seen to have its separate grillage. Fig. 182 shows the grillage for one of the heavier stanchions.

The base of a stanchion is treated as shown at Fig. 107. It is generally supported by a cast-iron base, to which it is bolted, as in Fig. 183, or it may be carried on to the grillage beams direct.

The cast-iron bases of the Ritz Hotel have an average

measurement of 3 feet square, with top and bottom plates $1\frac{3}{4}$ inch thick, and ribs $1\frac{1}{2}$ inch thick.

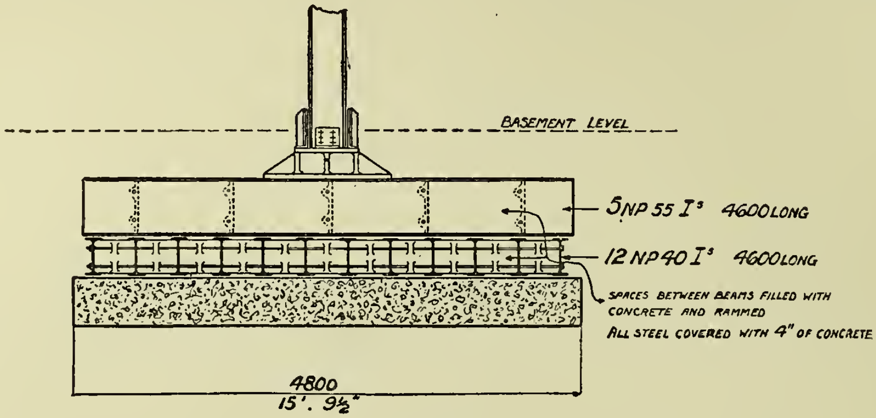


FIG. 182.

The grillage foundation is usually constructed as follows (see Fig. 184). First a bed of concrete, usually 18 inches thick, is laid of the necessary area of foundation. Upon this is placed a row of rolled steel joists, all bolted together with cast-iron separators between them. The spaces between the joists are then filled in with

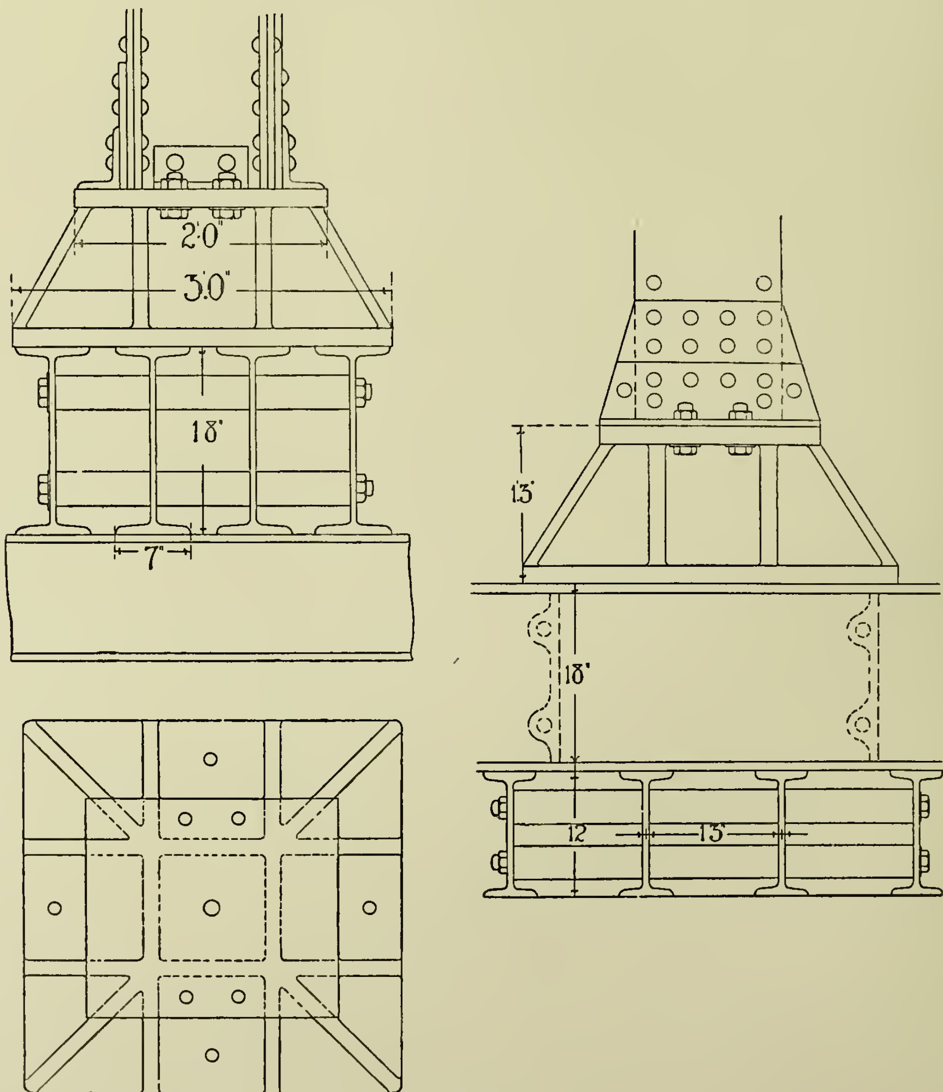


FIG. 183.

concrete, after which another row of joists is laid upon and at right angles to the first; and these are in turn filled in with concrete. Upon the top row of joists the cast-iron base is placed, which may, or may not, be bolted down; it is, however, often bolted down merely for the purpose of facilitating the erection of the stanchion. If the load upon the stanchion is very great, or if the resistance of the earth is poor, it may be necessary to employ three or four rows of joists in order to spread the load over a sufficiently large area. The joists of the grillage must not be placed so close together that concrete cannot be packed conveniently and thoroughly between them, whilst the aggregate of the concrete should preferably be of a fine gauge. For the sake of protection to the steel all metal surfaces should be covered with at least 1 inch of concrete, which should in turn be rendered in cement mortar.

Suppose it necessary to design a grillage foundation for a stanchion carrying a load of 288 tons, and having a cast-iron base 3 feet square. Assuming the earth to be capable of receiving a safe load of 2 tons per square foot, the necessary area of grillage = 144 square feet, or 12 feet square. The sizes required for the rolled steel joists may be calculated as follows. The projection of the beams of the bottom layer beyond those above them = $4\frac{1}{2}$ feet, with a total load of $4\frac{1}{2} \times 12 \times 2 = 108$ tons. This projecting portion may be considered as a cantilever. The bending moment produced in a cantilever = that produced in a beam of twice its length and with double its load (Chapter III.). Thus the joists of the lower layer must together be capable of supporting a distributed load of 216 tons over a span of 9 feet. This may economically be done by means of eleven lengths of a section 12×5 inches $\times$ 32 lbs. (see Table, p. 72).

In the upper layer the length of cantilever again = $4\frac{1}{2}$ feet, while the total load = 108 tons. Four joists may conveniently be placed under the cast-iron base, and the load on the projection of each = $\frac{108}{4} = 27$ tons. Then each joist must be capable of supporting a distributed load of 54 tons over a span of 9 feet. From the table above referred to the necessary section is seen to lie between 18×7 inches and 16×6 inches, and the larger would be selected. The bending moment, at some point under the cast-iron base, will be slightly greater than that found above; but against this may be put the extra resistance afforded by the concrete. The foregoing will be sufficiently accurate for practical purposes.

The above example is illustrated in Figs. 183 and 184. As there shown, the length of the joists need not be of the full width required for the foundation; but this will not affect the bending moment as allowed for above.

Concrete Rafts.—On very poor soil it is sometimes the practice to construct a raft or float of concrete over the entire site of the building. For this construction to be successful the raft must be made thick enough or strong enough to resist the upward pressure of the earth, taking each portion between two stanchions as a beam with evenly distributed load. (See Reinforced Concrete, Vol. V.)

Eccentric Foundations.—It is particularly important that the centre of pressure of the reaction of a founda-

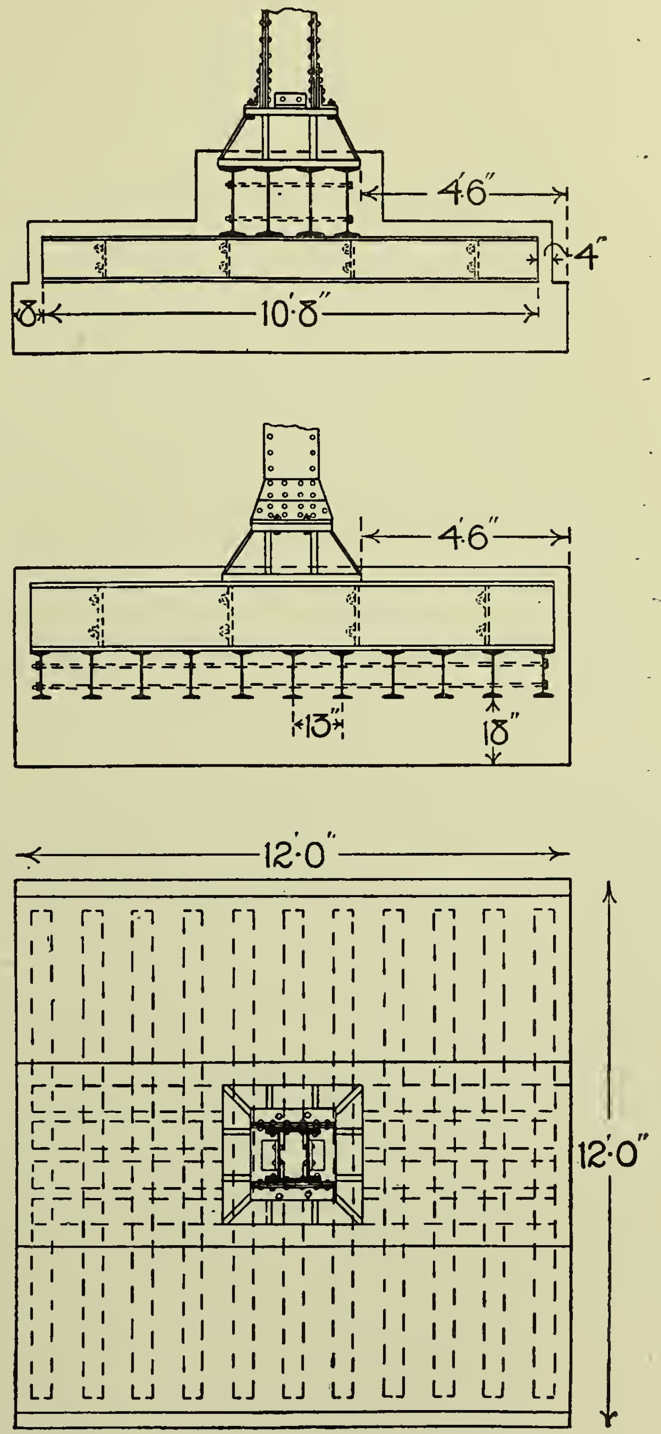


Fig. 184.

tion bed should be exactly in line with the centre of pressure of the load. If this be not the case a turning moment will be introduced equal to the total load $\times$ distance between the centres of pressure; and this B.M.

will have to be met by the resistance of the stanchion to bending, and by the addition of more metal to its section,—in the same way that allowance was made for eccentric loading (Chapters IX. and XV.).

Where an existing building stands alongside the building line, against which the proposed building is to be erected, it is often impractical to give the foundations an equal spread on all sides of the stanchion, while the foundations cannot be carried to any considerable depth without shoring and underpinning the adjoining wall. In this case the turning moment produced may be allowed for as mentioned above; but the B.M. will probably be considerable, and a better way of solving the difficulty will often be by means of

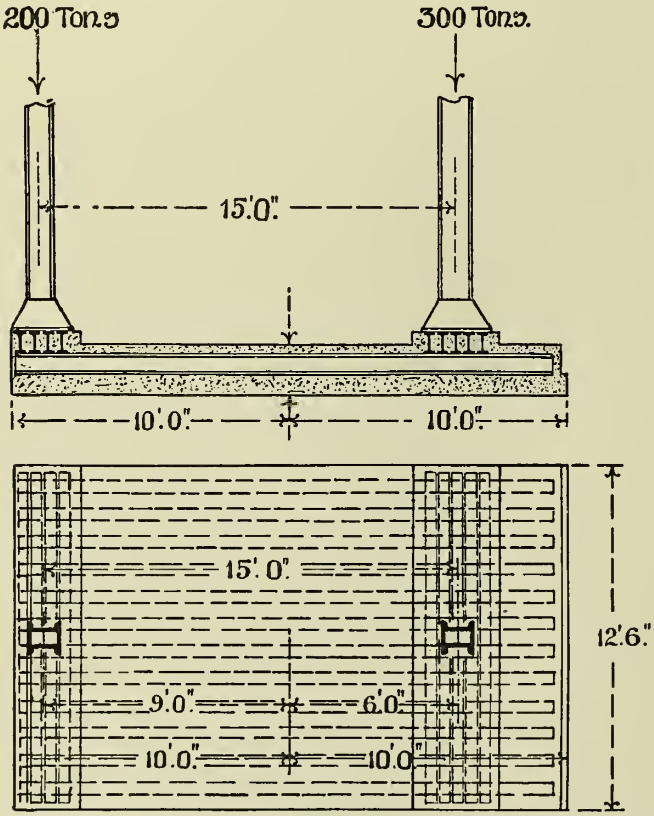


Fig. 185.

cantilever construction, or by the method of placing two or more stanchions upon a single grillage. The latter method may also be considered as a type of cantilever construction.

Single Grillage supporting two Stanchions. — Suppose that two stanchions, 15 feet apart, loaded with 200 tons and 300 tons respectively, are to be supported upon a single grillage (Figs. 185 and 186).

The resultant load = 200 + 300 = 500 tons, and the distance of its point of action from the left-hand stanchion $= \frac{300 \times 15}{500} = 9$ feet. Marking this point upon the plan, it must now be made the centre of the area of the grillage. Taking the safe load upon the foundation bed as 2 tons per square foot, the necessary area $= \frac{500}{2} = 250$ square feet.

If it be desired that the left-hand edge of the grillage shall come on a level with the edge of the stanchion base, the distance of edge from centre of pressure = say, 10 feet. The total length of grillage then $= 10 \times 2 = 20$ feet, and the necessary width $= \frac{250}{20} = 12\frac{1}{2}$ feet (see Fig. 185).

If it be desired that the edge of the grillage shall be on a level with the edge of the right-hand stanchion

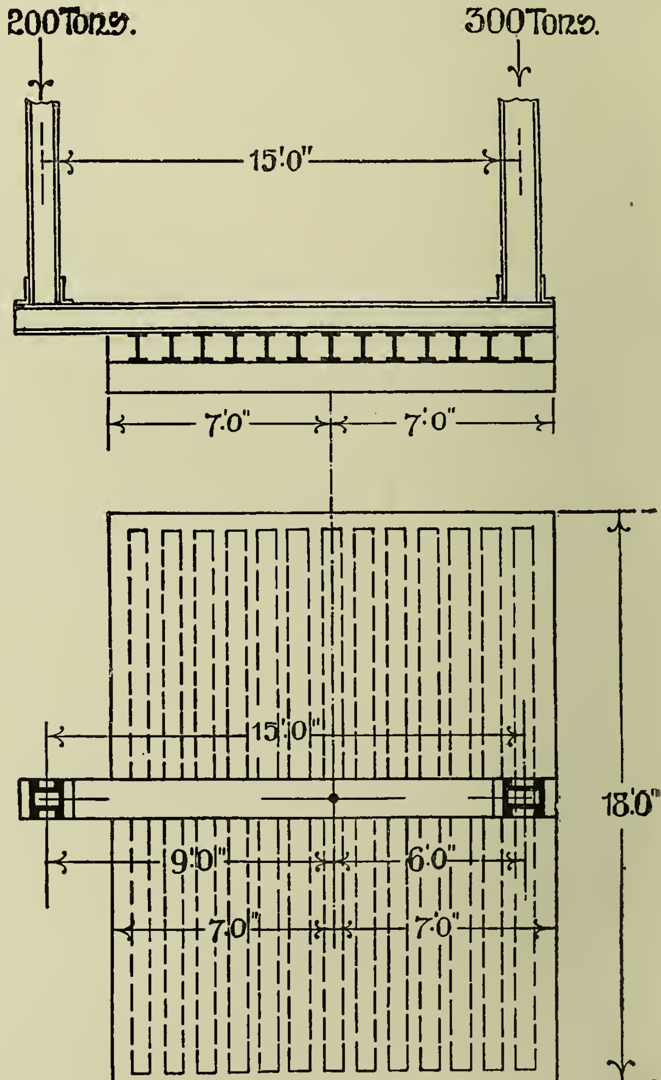
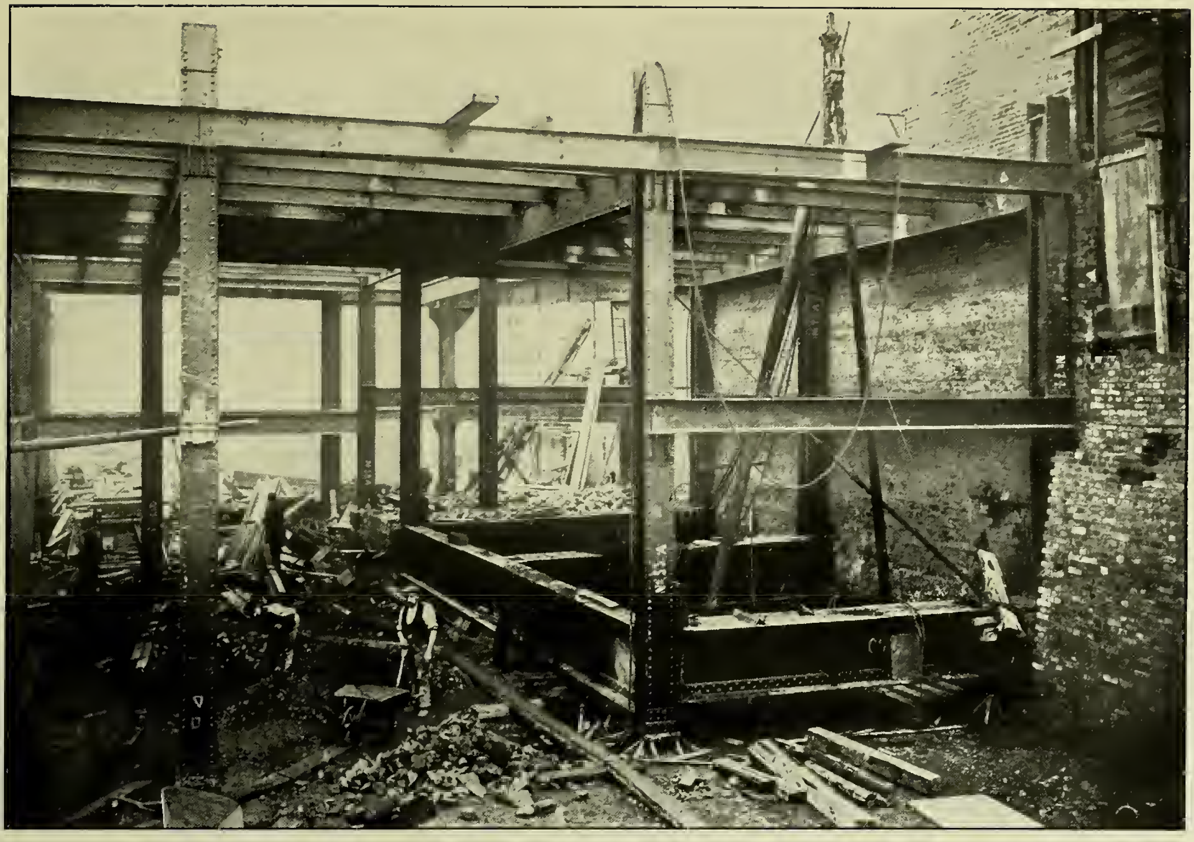


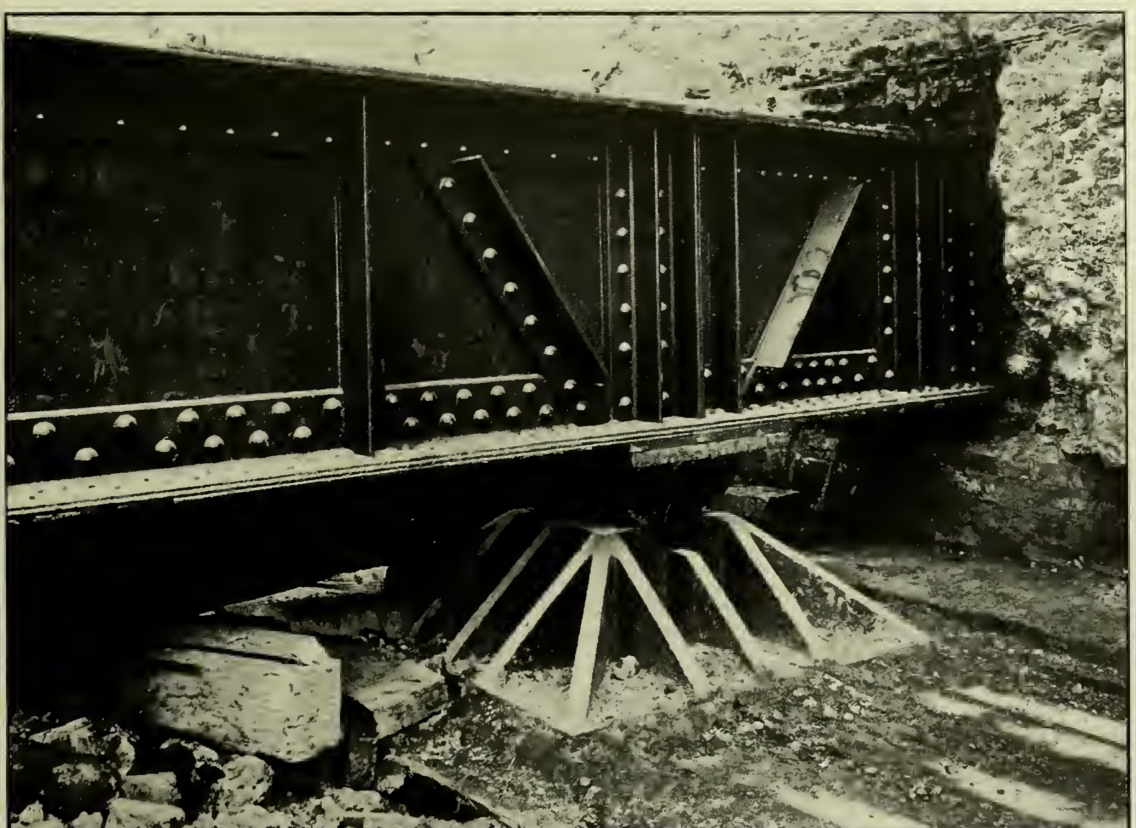
Fig. 186.

base the length of grillage $= 7 \times 2 = 14$ feet; and width $= \frac{250}{14} = 18$ feet (see Fig. 186). In this case the left-hand stanchion will have to be carried by a cantilever.

Girders or joists parallel to a line joining the centres of the stanchions may be calculated as simple beams with evenly distributed loads, while those at right angles to this line should be calculated as cantilevers, as was done in the example illustrated by Fig. 184.

The above principles will be useful, not only in the case where it is impossible to spread out the foundations





RITZ HOTEL, PICCADILLY, LONDON.
Cantilever and Grillage Foundations.

on account of an adjoining building, but also when the necessary areas of grillages would overlap if balanced evenly about the centres of the stanchions. The same principle may be applied to three or more stanchions, always finding the centre of pressure of the loads, and then balancing the necessary area about this point.

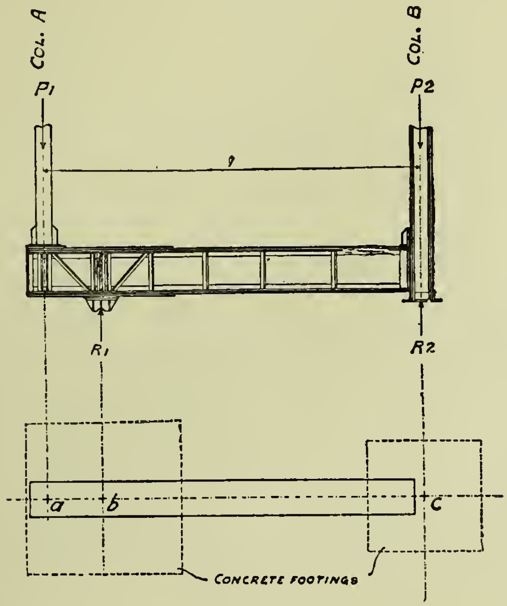


FIG. 187.

CANTILEVER CONSTRUCTION.—In the case where it is necessary to erect stanchions against an existing wall true cantilever construction will generally be more practical. By this means the excavations are kept clear of the adjoining wall, thus avoiding all need of shoring or underpinning. The Ritz Hotel gives what is

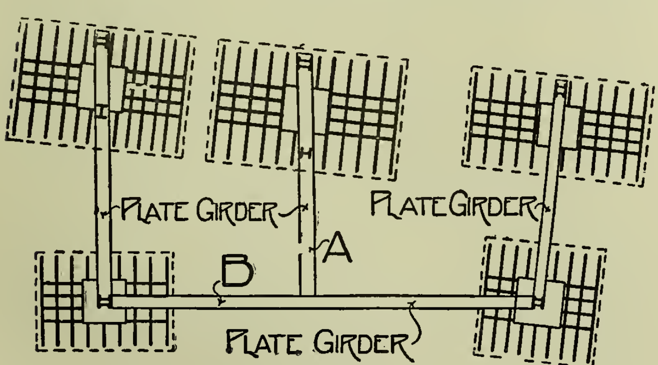


FIG. 188.

probably the first example of this construction in England, although the principle has been freely used in America. Figs. 187 and 188 give details of these cantilevers, which are again illustrated in the photographs on Plate IX. They are seven in number, and their position and further details may be seen on the plan, Fig. 181. It will be seen that two or three stanchions rest upon single plate girders, which are in turn carried by two cast-iron bases with their respective grillages. The load at the cantilever end of the girder may be seen in Plate IX. to be carried upon a steel pin. This will ensure that the load shall come upon the foundation at its centre, no matter what settlement may take place. The pin is not always provided in similar cases; but it is the only means by which an even bearing can be assured.

The B.M. diagram for a similar case is given in Fig. 189, being similar to that in Fig. 64; K. The maximum bending moment is over the pin, where it

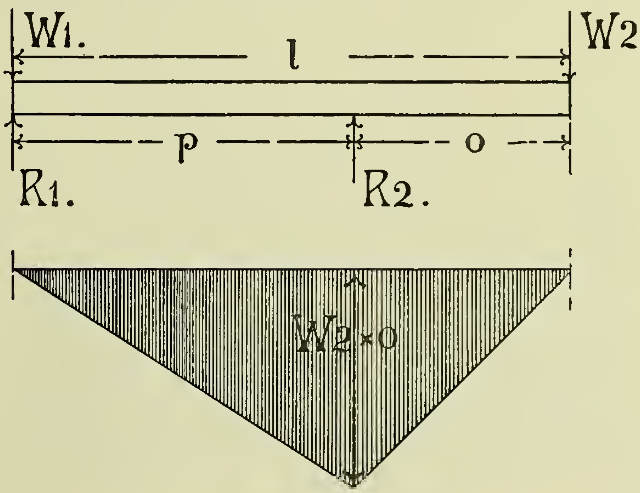


FIG. 189.

$=W_2 \times o$. Of the load W_1, part is required to resist the turning moment of W_2; putting W_1' for this part, $W_1' \times p = W_2 \times o$.

$$\therefore \ W_1' = \frac{o}{p} W_2.$$

$\therefore$ Reactions beneath $W_1 = R_1 = W_1 - \frac{o}{p} W_2$; and $R_2 = W_1 + W_2 - R_1$.

The areas of the grillages are then proportioned to R_1 and R_2.

At the lower extremity of the central girder A (Fig. 188) there is seen to be no stanchion, as this has had to be stopped and to be carried upon a cross girder in a floor above. This extremity must, however, be weighted down in some way, and this has been done by fixing it to the girder B, which in turn has been attached to the stanchion at either end. Fig. 190 represents girder A. As before, the amount of the load at the lower (now the left-hand) extremity necessary to balance the load at upper (now the right-hand) end $=W_3 = \frac{n}{m} W_4$; and this load is resisted by the cross girder B, which must be designed to carry it as a central concentrated load. Girder B then exerts upward forces upon the stanchions at its ends, each equal to $\frac{W_3}{2}$.

Therefore R_1 now becomes $W_1 - \frac{o}{p} W_2 - \frac{W_3}{2}$.

As the girders used in this construction have very heavy loads in comparison to their spans, particular provision is necessary to resist shear. Heavy stiffeners must be put immediately below or above the load or reaction, capable of carrying, as a pillar, at least half

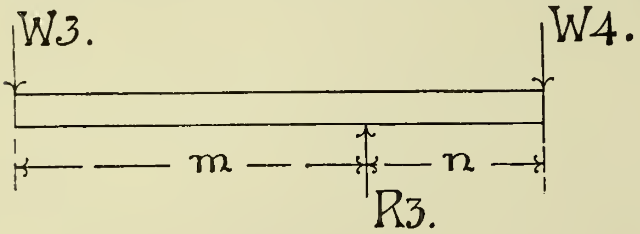


FIG. 190.

the amount of each. The cantilever girders of the Ritz Hotel are formed with 3 webs, as shown in Fig. 191. The diagonal stiffeners in Plate IX. should be noticed; also the staggered riveting introduced in the vertical arm of the angles for the purpose of resisting horizontal shear.

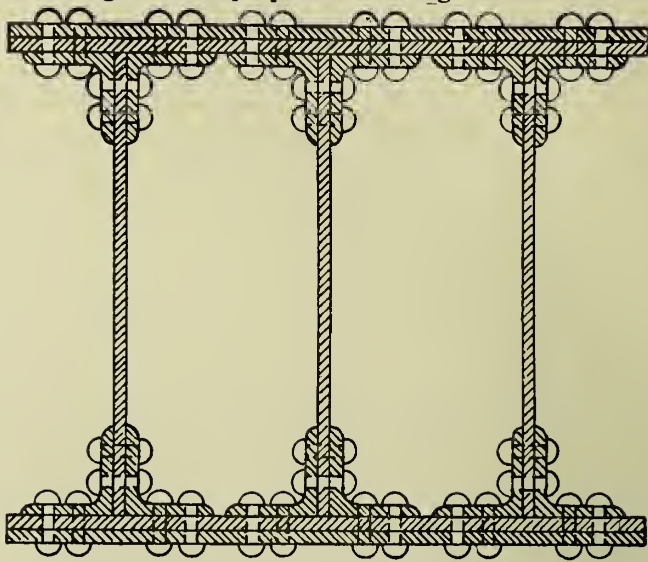
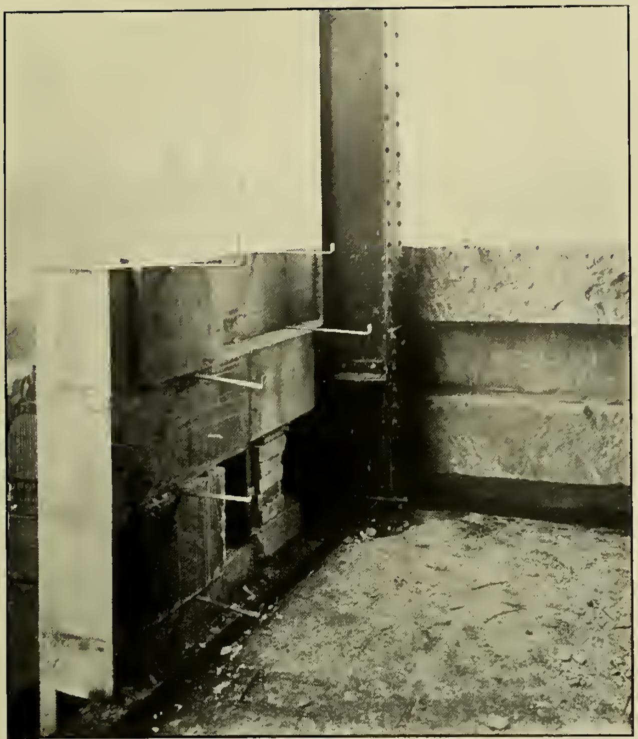
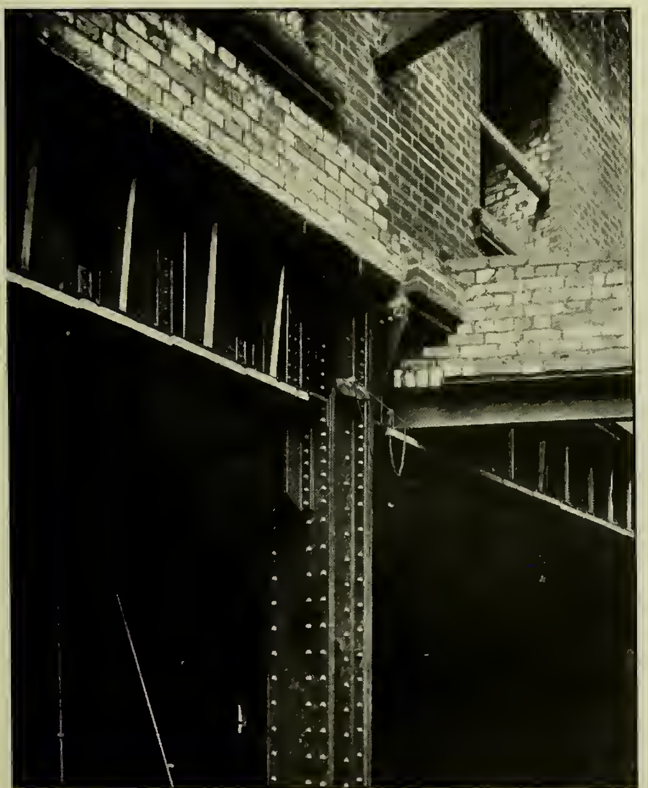


FIG. 191.



Connection of Stone Facing to Steelwork and Backing.

Casing of Steel Stanchions and Pipes.



Casing Underside of Girder, and Support of Walls.

Derrick Crane.

RITZ HOTEL, PICCADILLY, LONDON.

CHAPTER XVII

THE SUPPORT OF WALLS

THE necessity of protection from fire for all steelwork in the framing of a building is referred to in Part III. of this volume, where the qualities of various protecting materials are discussed. Practice as to the thickness of such protection varies considerably. Much depends upon the power of resistance of the material used, and it may be taken that the thicker the covering of a material of good fire-resisting qualities, the safer will be the structure. With a sufficiently prolonged high temperature even the thickest protection will be of little avail; but in a building in which the subject of fire resistance has received proper consideration the fuel will generally be insufficient to cause a very prolonged conflagration, so that covering of a moderate thickness will suffice. The thickness, then, should vary with the amount of inflammable material used or stored within the building; thus in a warehouse used to store inflammable goods the steelwork should be given greater protection than is necessary in a properly designed office building.

Apart from the question of fireproofing, a moderately thick protection in external walls will keep out damp and lessen the dangers of corrosion.

It seems reasonable to consider the following as minimum thicknesses of coverings when these consist of good brick or terra-cotta; but if of stone the thicknesses should be increased. For stanchions, 8 inches over parts facing the exterior of the building, and 4 inches when in the interior of the building; for girders, 4 inches when external, and 2 inches when internal.

As before stated, all walls are carried by girders at each floor level. The third illustration in Plate X. shows a plate girder carrying one of the back walls of the Ritz Hotel. It is seen to have a wide plate in the upper flange for the purpose of supporting the wall, which is strengthened by stiffeners taking the form of brackets. The slabs hung from the lower flange of the girder for its protection should also be noticed.

It is obviously a simple matter to support a wall upon a girder; but to protect the steelwork there must be brickwork, or some other protection, at the sides of the girder, which it must also support, while in the veneer construction, so much used in America, the protection to the under side of a girder must also be carried by it, being hung with iron hangers.

Fig. 192 is a section through a portion of the front wall and floor of the Ritz Hotel, showing how the protection about its sides is supported by the girder, the ashlar facing being fixed with the aid of iron cramps.

In America, sections such as those in Figs. 192, 193, are known as "Spandril sections," the wall between head and sill of two window openings being called a spandril.

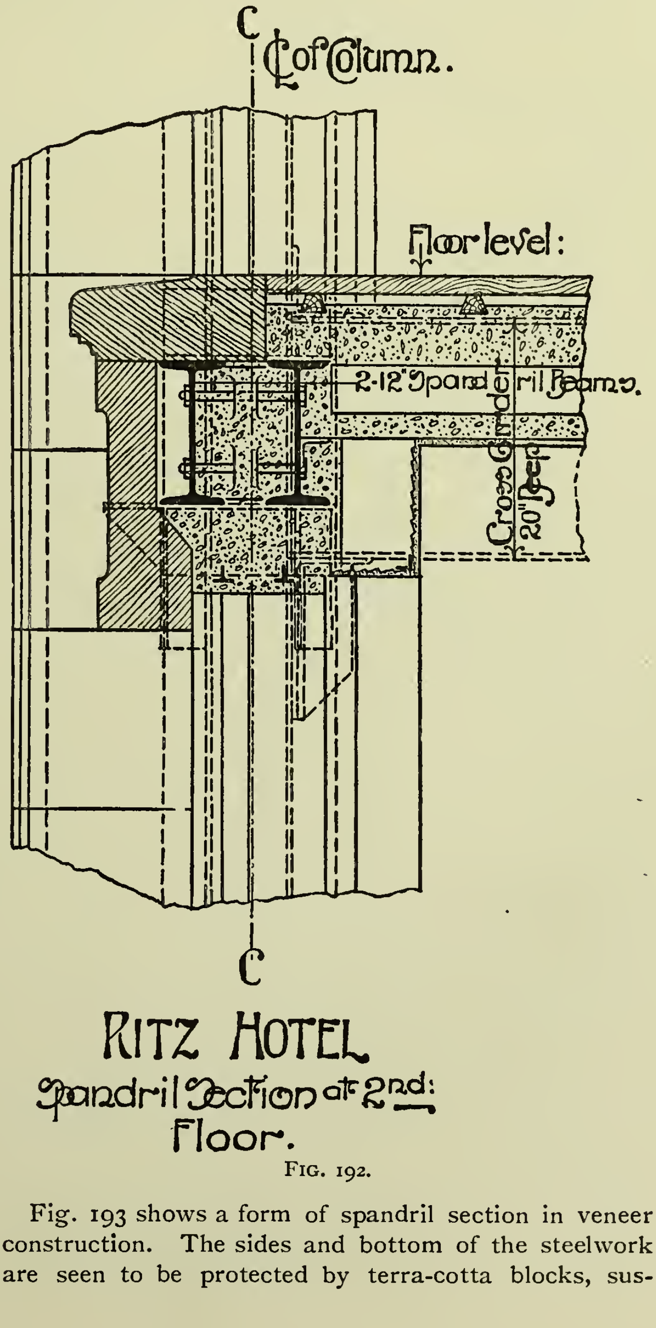


FIG. 192.

Fig. 193 shows a form of spandril section in veneer construction. The sides and bottom of the steelwork are seen to be protected by terra-cotta blocks, sus-

pended from the steelwork itself and shaped to fit upon its lower flanges, the window opening coming close up to its under side. In this connection terra-cotta is very largely used in America for the sake of its lightness, fire-resistance, and accommodation to repetition. The method of protecting the under side of a girder at this point varies greatly, and to do it thoroughly is always a difficult matter. The method of hanging the protection by metal hangers may seem

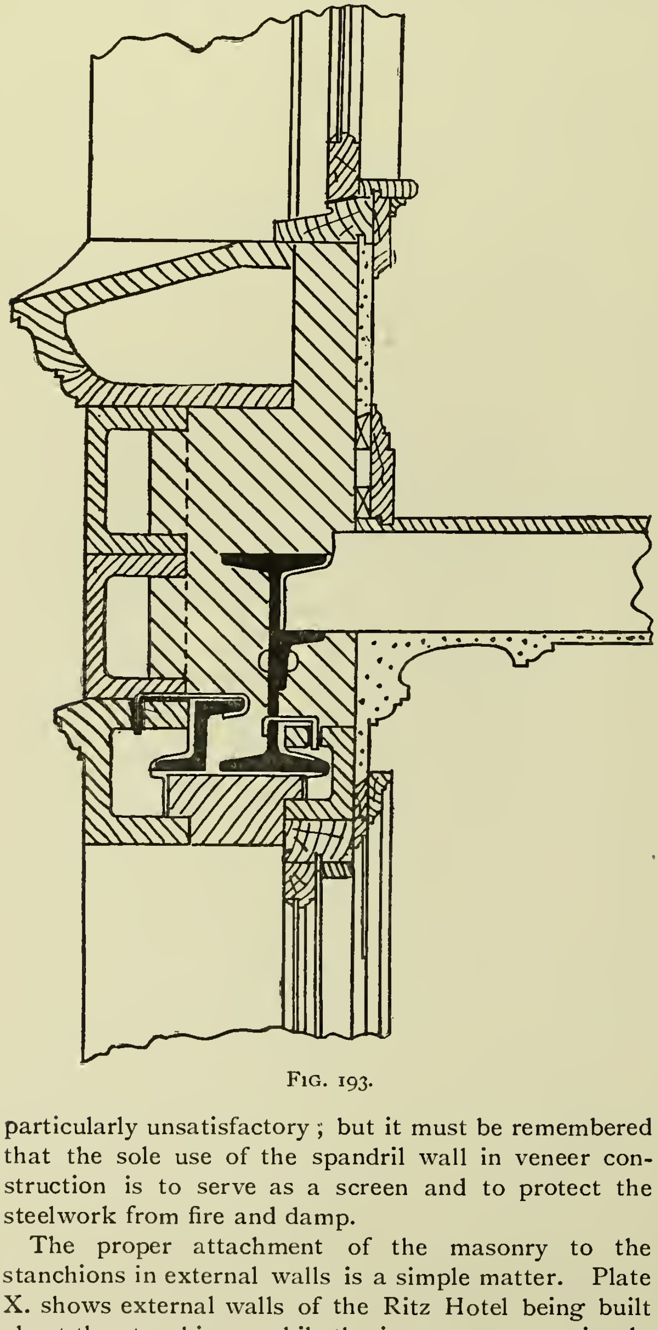


Fig. 193.

particularly unsatisfactory; but it must be remembered that the sole use of the spandril wall in veneer construction is to serve as a screen and to protect the steelwork from fire and damp.

The proper attachment of the masonry to the stanchions in external walls is a simple matter. Plate X. shows external walls of the Ritz Hotel being built about the stanchions, while the iron cramps previously mentioned may also be seen.

The protection of stanchions in the interior of a building as well as the protection of floor girders and joists will be considered in Part III.

Internal partitions should be light, thin, and fire-resisting (see Part III.). They should preferably come immediately over floor girders, but may be built upon any part of the flooring. In erecting some of the tall buildings of America many of the partitions were entirely omitted, to be set up afterwards in the positions required by the various tenants.

The Protection of Steelwork

As the steelwork forms the whole strength of a steel-framed building, it is evidently of primary importance that it should be protected as well as possible, not only from the effects of fire, but also against the attacks of corrosion. Were no steps whatever taken to prevent the latter evil, before a great number of years had passed rust would probably reduce the steelwork to a dangerous extent.

It has been fairly well demonstrated that steelwork *can* be efficiently, and apparently permanently, protected against corrosion; but it is a difficult matter in practice to ensure that principles, known to be efficient, are properly carried out.

Rust is formed in the presence of air, moisture, and acid, while if any one of this trio be eliminated rusting will be prevented. A small quantity of acid exists in even the purest air, and is particularly prevalent in the atmosphere of large towns.

Paint is the most common of preservatives, being intended to prevent any of the above rust-producing elements from reaching the metal. Good paint applied to a clean surface will protect it for a considerable number of years; but ultimately rust will be set up underneath the paint, which will be cracked and rendered more or less useless.

The life of a preservative coating of paint is particularly short where the structure is exposed to the sulphurous fumes of combustion, as in the case of station roofs and bridges.

There are certain bituminous preparations upon the market which are generally superior to paint, especially where injurious gases are to be met with.

Cement.—The most applicable protection to steel in building work, and one which is apparently almost perfect in its effect when properly used, is Portland cement. Although, when applied in thin layers, it may not be impervious to moisture and air, yet, being an alkali, it arrests and neutralises any acid present; besides which some chemical action seems to take place between the cement and iron, forming a protective coating to the latter. Examples exist of iron embedded in mortar in Roman times which, up to the present day, are still in good condition. Ironwork embedded in cement and standing in water has been found to be perfectly preserved. Ironwork in a slightly rusty condition, when imbedded in cement, has been found to be quite free from rust when eventually removed.

Lime might be supposed to have the same effect as Portland cement, and in some instances this has been

shown to be the case, but other examples have given distinctly unsatisfactory results. No evidence at present exists to show to what properties of lime mortar the protection, or want of protection, of ironwork is due, and in the meanwhile it will be best avoided in contact with steel.

Paint and Cement. — As stated above, Portland cement, even in thin layers, is a most efficient protection to steelwork, while it can be readily applied to the structure in setting the masonry about it. Having decided that Portland cement is an altogether admirable protection, it becomes a question whether steelwork should be painted before being embedded in the work.

As steelwork, after work has been completed upon it at the shops, is of necessity exposed to much weather before it is finally embedded, a coating of paint is a necessity before it leaves the shops. Again, in building the walls, etc. it is practically impossible to ensure that every inch of surface of steelwork is in contact with cement. Also it has been found that when cement mortar has been in actual contact with a painted surface the paint itself has apparently been perfectly preserved. Thus it is generally the practice to give all steelwork one coat of paint before leaving the shops, and one or two coats when erected in position. Nevertheless, if perfect contact with the cement could be ensured, it would probably be better to embed the metal free from paint, scraping the latter off before coating with cement.

Particular care should be taken in painting and protecting the joints, as it is at these points that the greatest likelihood of corrosion exists.

Cleaning.—In the process of rolling steel a black oxide of the metal is produced upon its surface. This oxide will scale off, and if it be not removed before paint is applied oxidation will take place under the paint and will eventually render it useless. The same results will also be produced by the ordinary red rust, which, if left upon a surface, will encourage the production of more. Thus before steel is painted it must be thoroughly cleaned. Steelwork is always specified to be thoroughly clean before paint is applied ; but it is most difficult to ensure this being carried out. The manufacturer and the architect may have different ideas as to the state of cleanliness necessary ; and when once the work is painted the existence of rust cannot be detected.

Steel may be cleaned by *Pickling*, by *Scraping*, and *Brushing* with wire brushes, or by the *Sand blast.*

Pickling consists of immersing in about a 10 per cent. solution of sulphuric acid to remove the scale. The metal is washed, and may then be immersed in a solution of carbonate of soda to remove all trace of acid, after which it is again washed in water, dried, and painted. This process is expensive ; also at the conclusion of the process the metal is wet and rust is able to start again, besides which it seems probable that the surface of the metal is permanently affected and rendered slightly brittle.

Cleaning by scraping is always most inadequate : plain surfaces, as a rule, are only partially cleaned, while possibly awkward corners will not even be touched.

The sand blast seems to be really efficient, leaving the metal thoroughly clean without the objections of the acid process, while the matt surface left is admirably adapted to receive paint. The apparatus consists of an air compressor and air chamber, the sand blast being delivered from a nozzle at the end of a flexible tube at a pressure of 5 to 25 lbs. per square inch. The sand projected against the metal will remove all paint, scale, and rust even from corners which are otherwise inaccessible. The process is undoubtedly thoroughly efficient, and has been largely used for cleaning small articles, castings, etc. ; but the cost has prevented its use upon structural work in England, although in America it has been satisfactorily employed for this purpose.

It is sometimes specified that as soon as metal is removed from the rolls it shall at once be cleaned and then dipped into a bath of linseed oil, and as soon as work upon it is complete it shall receive one coat of paint. This coating of linseed oil is most desirable, and should in any case be applied before plates are put together.

Concrete.—As cement is evidently such an excellent protection to steelwork, it is evident that cement concrete will also have protective properties, while in many cases it is preeminently suitable for use in conjunction with steel, notably in the construction of floors and grillage foundation, as well as in the case of reinforced concrete. For concrete to be an efficient protection it should be in perfect contact with the metal, and to secure this it will be well to use a fairly wet mixture, while the finer portions should be carefully packed about the metal surfaces.

If a large proportion of cement be used, and if particular care be taken to ensure it being in actual contact with the metal in all parts, it will probably matter little of what the aggregate consists ; however, it is not always possible to ensure this. Coke-breeze is an admirable aggregate for the concrete of floors on account of its lightness and fire-resisting properties ; but as regards its effect on the corrosion of steel it is to be treated with caution. It seems probable that coke-breeze does affect steel injuriously, and it will be well to avoid its use in such places as foundations, etc., where lightness is of little importance ; but in the case of floors its advantageous properties will well warrant its use, especially if care be taken that the mixture immediately around the steelwork is rich in cement.

Steelwork embedded in limestone, in certain cases, has been found to have become badly corroded. The reason for this is not clear, but limestone is evidently

to be regarded with suspicion either in the construction of walls or as the aggregate of concrete.

Galvanic Action.—Another consideration concerning the corrosion of steel is galvanic action. When two elements are in contact at one point, and connected at another point by slightly acidulated water, an electric circuit is produced, and a current will flow in it to the detriment of one of the elements. Elements are known as electro-positive and electro-negative in reference to one another. In a galvanic couple, such as is described above, the electro-positive element will be attacked while the other is preserved. With reference to iron, zinc is electro-positive, and therefore forms an excellent covering and protection. Copper, lead, carbon (soot), and oxygen are all electro-negative to iron, so that if one of these forms an electric circuit with iron a current is produced to the detriment of the latter. The ordinary rusting of iron may be put down to this cause.

As before mentioned, cramps are necessary to fix stonework, etc. to a metal structure ; but it is evident that copper should be avoided for the purpose for the reason just given. Zinc is evidently unsuitable, so that the choice turns to iron cramps, which at the same time will be the most economical.

Pipes.— The sides, between the flanges of the stanchions of a steel framework, form very convenient conduits up which to carry water pipes, gas pipes, soil pipes, etc. ; but this practice is to be strictly avoided, for, apart from the difficulty of repairing defects in the pipes at any future date, a defect may very likely go undetected, and the escaping gases will be most harmful to the steelwork.

Box Forms.—The box forms of stanchion and girder have the great disadvantage that their interiors cannot be painted when once the sides are put together, and the interiors will thus be liable to decay. In the case of the box stanchion, the interior may be protected by filling it with cement concrete, and this should always be done ; but it will militate greatly against the quickness of erection, and will be a considerable expense ; the practice is therefore rarely carried into effect. In the case of box girders the heavy load caused by filling them with concrete would be a great objection, so that this form is to be avoided wherever possible.

Masonry.—As a further protection to the steel framework the joints of the masonry should be kept well pointed, in order that moisture shall penetrate the wall as little as possible.

The Permanency of Steel Framed Buildings

Considering the perishable nature of steel, the permanency of such a structure as we have been considering is at present problematical. Construction of this sort is of comparatively recent birth, so that no very definite opinion can be formed ; however, the limited experience gathered hitherto tends to show that the life of such structures may be considerable. The framework being entirely embedded and covered up, frequent inspection and repainting are impossible, and the permanency of the building must be chiefly dependent upon the original protection of the metal. Even if it be demonstrated that the steelwork of one building is fairly permanent, it must not be taken for granted that that of another building which has apparently been similarly treated will be equally permanent, for experience goes to show that a very slight variation in conditions is sufficient to cause considerable variation in the extent of corrosion. It is therefore advisable that the steelwork should be examined at the end of, say, every fifteen or twenty years, by uncovering small portions of its surface ; but even if the portions thus uncovered are found to be in perfect preservation, it does not follow that some vital spot has not been dangerously attacked.

The desirability of steel frame construction need not be considered here, for it is purely a question of commercial needs and investments. If by using this construction a good return may be obtained upon the capital expended, this will be sufficient cause to warrant its adoption ; also the planning suitable to buildings of to-day may be quite inadequate long before the strength of the structure has been seriously reduced.

Testing.—For a specification of steel the publications of the Engineering Standards Committee may be referred to. Clauses as to tests and the extent of tests will naturally vary with the importance of the structure. When only a small quantity of steel is to be obtained extensive tests are out of the question, and the manufacturer must be trusted to supply material equal to that specified, while the architect merely satisfies himself with the quality of the workmanship.

When a large quantity of steel is required, an extensive system of testing and inspection may be carried out. The metal as it comes from the rolls is marked with the number of its particular cast. Test pieces are prepared from metal from each cast, and are tested by, or in the presence of, the architect or his representative. Care, however, must be taken that the pieces tested actually come from the right casts, and to ensure this some private stamp should be put upon the selected piece of metal before it is cut off to form a test piece.

A middle course is to have test pieces taken at random from the material when it has all been rolled ; but in this case, if any of the metal be found defective, the amount of defective metal cannot be known without testing every piece unless the number of the cast has been put upon each.

Having tested the materials at the mills, steps must be taken to ensure that the same metal is actually used at the shops for the manufacture of the forms ordered, and for this purpose some mark should be stamped upon all the metal while it is at the mills.

Cast-iron test bars should be cast with each important

casting, forming a branch from it which is afterwards broken off and tested. In order to see that a cast-iron column is of even thickness, small holes about $\frac{1}{8}$ inch diameter may be drilled in its walls. The

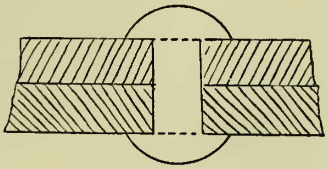


Fig. 194.

thickness in no part should be more than 15 per cent. less than specified.

Workmanship.—Most of the steelwork that has been considered in this Volume when in position will be out of sight, or at any rate at some distance from the eye.

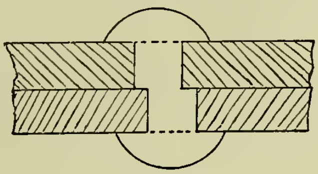


Fig. 195.

It is therefore perfectly unnecessary and extravagant to design or specify such refinements as accurately curved gussets and finely tooled edges, so long as any such points will in no way affect the strength of the design ; however, such work as is necessary should be of the best.

Rivet holes, whether punched or drilled, must be

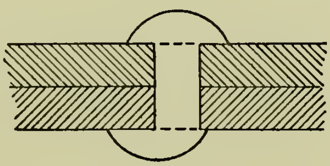


Fig. 196.

truly opposite one another. Fig. 194 shows the appearance of good riveting, while Fig. 195 shows the effect upon rivets if holes are not accurately to template. When holes are first punched and then rimered to size, the rimering should take place with

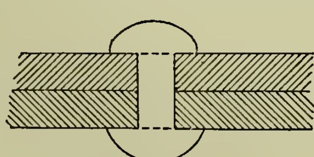


Fig. 197.

the plates fixed together in the position which they will occupy, in order to ensure that the holes are concentric throughout.

Fig. 196 shows one result of careless riveting. Fig. 197 shows the result of using a rivet with too little metal in its shank, while Fig. 198 shows a defective head due to this same cause, and in which the cupping tool has been made to cut into the plate. Fig. 199 shows the result of an excess of metal in the rivet, but beyond its appearance this defect is harmless.

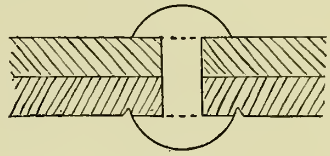


Fig. 198.

The shank of a rivet must fill the rivet hole, and should be quite tight. Loose rivets may be detected by tapping with a hammer.

In designing steelwork as little field riveting as

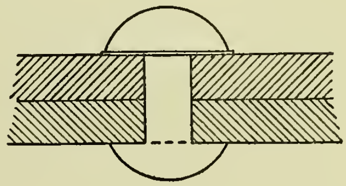


Fig. 199.

possible should be allowed, as this is more expensive and often less efficient than shop riveting.

The heat to which rivets are raised must be left almost entirely to those responsible for it. Care should, however, be taken to see that the metal is not "burnt,"—that is to say, raised to such a temperature that its strength is partially or quite destroyed. It is, however, difficult to detect whether a rivet has been burnt when once it is driven. Steel rivets will not withstand so great a heat as those of wrought-iron. The most satisfactory heat depends upon the method of riveting: in hand riveting a moderately high tem-

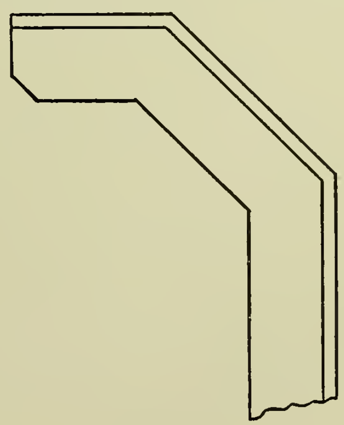


Fig. 200.

perature is necessary, and the rivet should be uniformly hot in order to completely fill the hole and to bring the plates together on cooling ; in hydraulic riveting with great pressure the heat should be less, while if possible the point only should be at welding heat.

In forging a stiffener of such a shape as in Fig. 200 it is evident that the surplus metal of the web must be taken up by thickening it at each bend. Another way is to cut the web as in Fig. 201, welding the

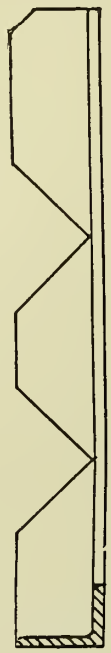


Fig. 201.

edges together when the bend is formed. However, the quality of the resulting weld is a doubtful matter, and the first method is much to be preferred.

In forming a bend in the reverse direction (Fig. 202) it must be remembered that the thickness of web, and therefore its strength, will be decreased at this point. Such a bend may again be effected by cutting the web and welding in a piece of metal as shown dotted ; but, as just stated, welds of this description are to be distrusted.

Erection.—Before despatching steelwork to the site all stanchions, girders, etc. should be numbered distinctly in paint according to the number given on

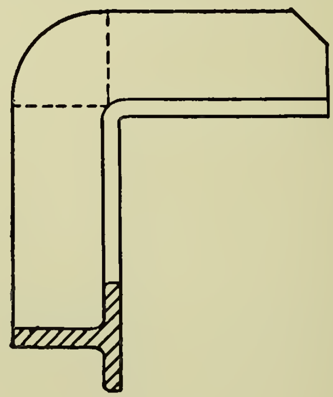


Fig. 202.

the drawings. Where girders are not exactly symmetrical in their connections clear directions as to their position should be painted upon them.

Plate X., besides illustrating the various methods of casing and protecting the steelwork, also shows the form of crane or derrick used in the erection of the steelwork of the Ritz Hotel. Where available, electric power is used with increasing success for this purpose.

CHAPTER XVIII

WOODEN BEAMS AND PILLARS

(*Contributed by H. Y. Margary*)

Beams.—The theory relating to beams of wood is identical with that relating to beams of steel, save that with steel the section of the beam may be varied, the amount of metal being apportioned to the varying stresses at different points along the beam ; while with wood the beam is almost invariably made of uniform section throughout its length. The reader should therefore make himself familiar with the matter contained in Chapters II., III., and IV.

It has been explained in connection with Fig. 50 (Chapter II.) that the moment of resistance of a beam of rectangular section is $fo\frac{bd^2}{6}$, where fo is the modulus of rupture of the material of which the beam is made, b the breadth, and d the total depth of the beam. It has also been shown that if a beam is sufficiently strong to resist the load placed upon it the *Bending moment must be equal to the Moment of resistance*,—or, as may be expressed in simpler notation—

$$BM = MR.$$

$$\text{Hence } BM = fo\frac{bd^2}{6}$$

If a constant K be substituted for $\frac{fo}{6}$ the above formula becomes

$$BM = Kbd^2.$$

This formula should be remembered, as it is universally true for all beams.

By putting in the values of the bending moments in the simpler cases the following results are obtained :—

Cantilever with end load—

$$BW \,.\, l = Kbd^2. \qquad \therefore BW = \frac{Kbd^2}{l}.$$

Cantilever with distributed load—

$$\frac{BW \,.\, l}{2} = Kbd^2. \qquad \therefore BW = \frac{2\,Kbd^2}{l}.$$

Beam with central load—

$$\frac{BW \,.\, l}{4} = Kbd^2. \qquad \therefore BW = \frac{4\,Kbd^2}{l}.$$

Beam with distributed load :—

$$\frac{BW \,.\, l}{8} = Kbd^2. \qquad \therefore BW = \frac{8\,Kbd^2}{l}.$$

It should be carefully noted that in the above formula BW, or the breaking weight, is considered, and not the safe load ; because K involves the modulus of rupture and not the modulus of safety. The values of K have been found by experimenting upon specimens of timber of various kinds, and are given in the table at the end of this chapter.

Knowing the safe load to be carried by a beam, the BW is found by multiplying the safe load by some suitable factor of safety. The factor of safety to use with wooden beams varies according to different authorities, but $\frac{1}{5}$ is the factor in most general use ; for it has been found that a strain greater than $\frac{1}{5}$ of the breaking weight causes undue deflection and in course of time produces a permanent set. This factor of $\frac{1}{5}$ applies to fixed loads, a factor of $\frac{1}{7}$ is taken for moving loads.

The breaking weight and the constant K must both be expressed in cwts., since the loads on beams are most conveniently calculated in cwts.; and b, d, and l must be expressed in inches.

The application of the formula will be explained by the following example. Suppose a floor bearing a load of $1\frac{1}{4}$ cwt. per square foot is to be supported upon walls 14 feet apart, the joists being of yellow pine and spaced at 12 inches centre to centre. What size joists must be used ?

The load carried by each joist is a distributed one of $1\frac{1}{4} \times 14$ cwts. $= 17\frac{1}{2}$ cwts. Now with joists it is desirable to make the depth as great as possible, consistently with safety against buckling. It is also necessary that the breadth should be sufficient to resist splitting when the floor brads are driven in, and for the latter reason joists should not be less than 2 inches in breadth.

In the case under consideration.

$$BW = \frac{8Kbd^2}{l}.$$

$$\therefore \text{Safe load} \times 5 = \frac{8 \times 10 \times 2 \times d^2}{12 \times 14}.$$

$$\therefore d^2 = \frac{35 \times 5 \times 12 \times 14}{2 \times 8 \times 10 \times 2}$$

$$= \frac{735}{8}$$

$$= 91.9.$$

$$\therefore d = 9\tfrac{1}{2} \text{ inches approximately.}$$

If large beams are required, where the width is not governed by such practical considerations as were mentioned in the case of floor joists, the strongest

beam will be obtained by making the ratio of breadth to height equal to $\frac{5}{7}$, so that the formula becomes

$$\mathrm{BM}=\frac{\mathrm{K}5d^3}{7}.$$

In working out the sizes of beam where the loading is at all unequally distributed throughout its length, it is necessary first of all to find the Bending moment as explained in Chapter III. The formulæ for the four simple cases quoted above are quite easily remembered. In Fig. 203 the case of an oak beam with a cantilever at one end is shown where the loading is unequally distributed, as indicated in the diagram.

The BM at B due to end load alone is equal to $8\times5\times12=480$ inch-cwts. The BM at B due to the

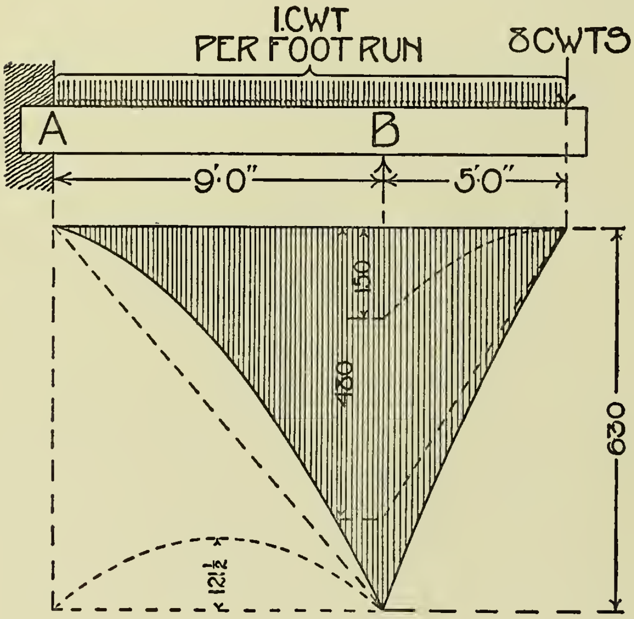


Fig. 203.

distributed load on the cantilever alone is equal to $\frac{5\times5\times12}{2}=150$ inch-cwts. The BM at the centre of AB due to distributed load on AB, if considered separately, is equal to $\frac{9\times9\times12}{8}=121\frac{1}{2}$ inch-cwts. A BM diagram is now drawn as shown in Fig. 203 and explained in Chapter III., where it will be seen that the maximum BM is at A, and is equal to $480+150=630$ inch-cwts.

$$\text{Now BM}=\frac{\mathrm{K}5d^3}{7}.$$

$$\therefore\ \text{Safe BM}\times\text{Factor of safety}=\frac{\mathrm{K}5d^3}{7}.$$

$$630\times5=\frac{15\times5\times d^3}{7}.$$

$$\therefore\ d^3=\frac{630\times5\times7}{15\times5}$$

$$=294.$$

$$\therefore\ d=6\tfrac{3}{4}\text{ inches approximately.}$$

$$b=\tfrac{5}{7}d=4.82,\text{ say 5 inches.}$$

A little experience of bending moments would have made it evident that the maximum bending moment occurred at B; but in the majority of cases where irregular loading is considered it is not clear where the maximum bending moment comes, and then it is generally advisable to draw a bending moment diagram from which to scale the maximum bending moment to apply to the formula.

There are unfortunately two formulæ for calculating the strength of wooden beams in common use, known respectively as the $\mathrm{K}bd^2$ and the $\mathrm{C}bd^2$ formulæ. These formulæ give exactly the same results, as they depend upon the same theory, yet there are no two formulæ in theoretical construction which give rise to so much confusion. For this reason both formulæ are explained here, in the hope that the minds of many readers will be relieved of the confusion resulting from these formulæ.

The moment of resistance of a beam of the section shown at A, Fig. 204, as already explained, is

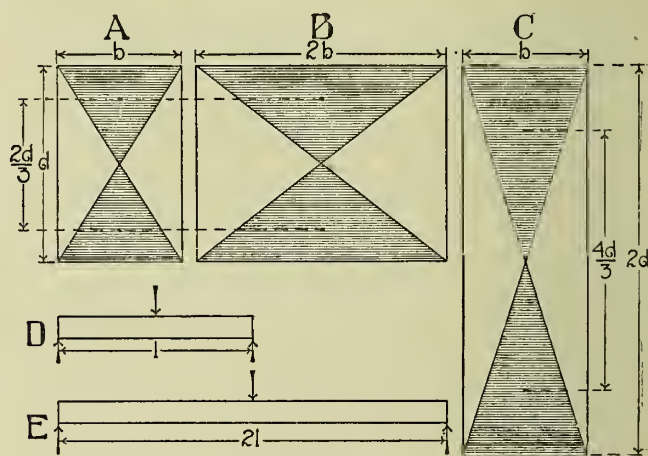


Fig. 204.

equal to $fo\frac{bd^2}{6}$. If the breadth of the beam be doubled, as in B, Fig. 204, the effective area is also doubled, and the moment of resistance becomes $fo\frac{bd^2}{3}$. That is to say, by doubling the breadth of the beam its strength has also been doubled. The strength of a beam therefore varies directly as its breadth. Also at C, where the breadth remains the same as in A, and the depth has been doubled, it will be seen that not only has the effective area been doubled but the effective depth has also been doubled, and the moment of resistance becomes $fo\frac{4bd^2}{6}$. That is to say, by doubling the depth of the beam the strength has increased fourfold. The strength of a beam therefore varies directly as the square of its depth. At D and E, Fig. 204, two beams are shown, E being double the length of D. It is clear that the maximum bending moment in the case of E is double that in the case of D under the same load, and the strength of E is half

that of D. The strength of a beam therefore varies inversely as the length.

The above facts, expressed in mathematical notation, become

$$\left.\begin{array}{l} BW \propto b \\ BW \propto d^2 \\ BW \propto \frac{1}{L} \end{array}\right\} \text{Where } b, d, \text{ and L are respectively the breadth, depth, and span.}$$

Hence $BW = C\frac{bd^2}{L}$, where C is a constant.

In view of these facts, experiments have been carried out upon samples of timber 1 inch square, which were supported upon bearings 1 foot apart, and loaded at the centre until they broke, in order to ascertain a value for C.

This equation $BW = C\frac{bd^2}{L}$ forms the well known formula for calculating the strength of wooden beams with central loads, and it should be remembered that b and d are in *inches* while L is in *feet*, because these were the units used in the experiments when finding values for C. BW must be expressed in cwts., because C is usually given in cwts. for the reason explained in connection with the Kbd^2 formula. The same factors of safety are used with the Cbd^2 formula as with the Kbd^2 formula.

The formulæ for the simple cases of beams are as follow:—

		Relative Strength.
For a cantilever with end load,	$BW = C\frac{bd^2}{4L}$	1
For a cantilever with distributed load . . .	$BW = C\frac{bd^2}{2L}$	2
For a beam with central load,	$BW = C\frac{bd^2}{L}$	4
For a beam with distributed load	$BW = 2C\frac{bd^2}{L}$	8

The formulæ for cantilevers and beams with distributed loads follow directly from the formula for beams with central loads, from the fact that the *bending moments*, and hence the relative strengths, of beams of the same size are in the ratio shown above.

If a beam is loaded unevenly the maximum bending moment must be found, and also the load which would give the same maximum bending moment when placed at the centre of the beam.

The Kbd^2 formula is decidedly preferable to the Cbd^2 formula, because it is more scientific, inasmuch as all the lineal dimensions are of the same denomination, and in every possible case which has to be calculated the same method is employed, namely, that of equating the Bending moment with Kbd^2. With the Cbd^2 formula there is the additional trouble of finding the central equivalent load in ordinary cases, while its application to a case similar to that taken in Fig. 147 is very troublesome.

It should be noticed that both the formulæ are really identical, for if we take the case of a beam with a central load the BW given by the Kbd^2 and Cbd^2 formulæ respectively are—

$$\frac{4\,Kbd^2}{l} \text{ and } \frac{Cbd^2}{L}.$$

$$\text{Now } \frac{4\,Kbd^2}{l} = \frac{4 \times 3\,Cbd^2}{12\,L} \text{ (see table of values of K and C)}$$

$$= \frac{Cbd^2}{L}.$$

In a similar manner the identity may be proved in all other cases.

Deflection of Beams.—Any load placed upon a beam will cause it to deflect, and it is often necessary to test a beam in order to ascertain whether the deflections produced by a given load is not excessive. For this purpose the formula shown on page 65 must be used, namely—

$$D = m.\frac{Wl^3}{EI}.$$

I for rectanglar sections being $\frac{bd^3}{12}$, the formula for wooden beams becomes

$$D = m.\frac{Wl^3 \times 12}{Ebd^3}.$$

The values of E for various varieties of timber are given in the table at the end of this chapter, while the values for all the other terms are the same as given in Chapter IV.

PILLARS.—The formula in most general use for calculating wooden pillars is that known as Gordon's formula, which may be stated as follows:—

$$\text{Breaking weight in tons} = \frac{fA}{1 + c\left(\frac{l}{d}\right)^2}.$$

The significance of the terms in the formula being the same as shown in connection with the same formula on page 90—

$$c = \tfrac{1}{250} \text{ for rectangular sections.}$$
$$= \tfrac{1}{187} \text{ for circular sections.}$$

The factor of safety to use in the above formula is $\frac{1}{10}$.

Wooden pillars are, as a rule, bedded flat,—that is to say, their ends are squared, and butt square against horizontal timbers at the head and base, and are spiked or pinned to keep them from being knocked out of place. For this reason the above formula has been chosen, as it applies to the most general case, and will also, with the slightest modification, apply to cases where the ends are differently treated.

The degree of fixity of the ends of a pillar affects its strength considerably. The ends of a pillar may be treated in four ways—(1) It may be firmly fixed at both ends; (2) it may be rounded or hinged at both ends; (3) it may be fixed at one end and rounded at the other; (4) it may be fixed at one end and free at the other. See Figs. 97 to 100. In practice it is impossible to absolutely fix the end of the pillar,

and it is very seldom that the ends are rounded off; but conditions approximately similar will be met with.

A pillar rounded at both ends is as flexible as a pillar of the same diameter fixed at both ends and double the length, and its strength might be expected to be the same. This conclusion was verified by the experiments of Mr. Hodgkinson, who also found that the strength of a pillar fixed at one end and rounded at the other was approximately a mean between the strengths of two pillars of the same diameter, one fixed at both ends and the other rounded at both ends. In practice the ends of pillars are never absolutely fixed, as were the specimens used in Mr. Hodgkinson's experiments, but are merely flat-bedded, so that the strength lies somewhere between that of pillars firmly fixed and of those rounded at both ends. When pillars are very long—*i.e.* over 30 diameters—and no great care has been taken to fix the ends firmly, they should be considered as having rounded ends.

The formula must therefore be modified to suit the various methods of fixing the ends, as shown on page 91.

In the above formula there is only one unknown quantity to be found. If it is desired to find the load which will be safely carried by a post of given length and given breadth and thickness, then W—*i.e.* safe load × 10—is the unknown quantity. If, as is more usual, it is necessary to find the section of a pillar of given length required to support a given load, then d is the unknown quantity. The value of f must be taken from the table below. A is obtained in the following manner. If it is proposed to cut the pillar from a piece of timber of given thickness t, then $A = td$; but the strongest section of a pillar is square, so that, should it be considered advisable in any particular case to use a square pillar, A becomes $= d^2$.

Table of Mean Crushing Weights per Square Inch of Short Specimens of Timber in the Direction of the Grain:—

	Tons
Ash	3.50
Beech	3.80
Birch	2.72
Elm	3.00
Fir (Norway Spruce)	2.50
Oak, English and Dantzic	3.20
,, Canadian	3.68
,, Amercian white	2.97
,, ,, red	2.68
Pine, Northern Memel	2.90
,, Dantzic	2.95
,, Riga	2.30
,, American white	1.85
,, ,, red	2.20
,, Pitch	3.02
Teak	3.02

The crushing weights of timbers shown in the above table are the means of the results of a number of experiments. It has been shown by the experiments of Rondelet and others that where a pillar is not more than 8 diameters high it will not fail under the action of an axial load less than that required to crush a short specimen of the same material of the same sectional area; and in practice, where safe loads only are considered, it is usual to calculate for direct compression only in all "short" pillars—*i.e.* pillars not more than 10 diameters in height.

The formula for short pillars, therefore, is—Breaking weight = breaking weight in tons per square inch × sectional area in square inches. Or, $W = fA$. The factor of safety to be used in this case is $\frac{1}{4}$.

Pillars of greater height than 10 diameters, up to 30 diameters, or, as they are generally called "long" pillars, fail partly through direct compression and partly through bending. This bending is due partly to the difficulty of loading the pillar absolutely axially, and partly due to the difference in elasticity of the wood in various parts of the pillar.

Gordon's formula therefore only applies to long pillars, and it has been stated above that the factors of safety usually used in connection therewith is $\frac{1}{10}$; but it is clear if there be two pillars of the same sectional area, one of say 9 diameters and one of 11 diameters in height, calculated with the factors of safety of $\frac{1}{4}$ and $\frac{1}{10}$ respectively, that the one will appear to be $\frac{10}{4}$, or $2\frac{1}{2}$ times as strong as the other. This is absurd, and it is better, therefore, to use a factor of safety for long pillars increasing with the length. A satisfactory factor of safety to use with long pillars is given by the following formula:—Factor of safety $= 4 + .2\frac{l}{d}$.

When it is required to find the load which will be carried by a pillar of given dimensions, $\frac{l}{d}$ of course is known; but in the more usual case of finding the sectional area of pillars of given length to carry a given load the diameter should be calculated with a factor of safety of $\frac{1}{10}$, and if it works out less than $\frac{1}{30}$ of the height it should be recalculated with the factor of safety given by the above formula,—d being the value as found by using a factor of safety of $\frac{1}{10}$ in Gordon's formula.

For examination purposes it is quite impossible for the average brain to hold a complete table of crushing weights, but there should be no difficulty in remembering fir as 2.5, pine as 3.0, and oak as 3.5 tons per square inch respectively.

It should be noted that the soundest wood should be chosen for pillars, with the least possible number of flaws and the straightest possible grain. The timber should also be dry and well seasoned, for if it is wet or green its strength, according to Hodgkinson, is only half that of well-seasoned timber.

By way of example, let us find the section of a square pillar of English oak 12 feet long with the ends flat-

bedded which will safely carry 6 tons. By substituting values in Gordon's formula—

$$6 \times 10 = \frac{3.20 \times d^2}{1 + \frac{(12 \times 12)^2}{250 d^2}}.$$

$$\therefore d^4 - \frac{75}{4} d^2 = 1555.2.$$

Add half the square of the coefficient of d^2 to both sides of the equation, thus—

$$d^4 - \frac{75}{4} d^2 + \left(\frac{75}{8}\right)^2 = 1555.2 + \left(\frac{75}{8}\right)^2.$$

Hence, by extracting the square root of each side

$$d^2 - \frac{75}{8} = \sqrt{1555.2 + 87.9}.$$

$$\therefore d^2 = 49.38.$$

$$\therefore d = 7 \text{ inches approximately.}$$

In this case $\frac{l}{d} = 20$ approximately. Hence a factor of safety should be $4 + .2 \times 20 = 8$ and the diameter may be recalculated with the value.

Had the pillar been greater than 30 diameters the value of d would have had to be recalculated as a pillar with both ends rounded, as stated above, substituting $3\,c$ for c in the formula.

Table of Values of K and C, and Modulus of Elasticity.

Material	$K = \frac{fo}{6}$.	$C = \frac{fo}{18}$.	Modulus of Elasticity.
	Cwts.	Cwts.	Cwts.
Ash, English	19	$6\frac{1}{3}$	14,640
Beech	13	$4\frac{1}{3}$	12,000
Birch	17	$5\frac{2}{3}$	....
Elm	7	$2\frac{1}{3}$	11,960
Fir (Spruce)	12	4	....
Oak, English	15	5	12,680
Pine, Red	13	$4\frac{1}{3}$	16,520
,, Yellow	10	$3\frac{1}{3}$	13,840
,, Memel	12	4	14,280
,, Pitch	15	5	16,960
Teak	19	$6\frac{1}{3}$	21,420

The modulus of elasticity is given here in cwts., because in considering wooden beams W and C are generally expressed in cwts.

PART III

FIRE-RESISTING CONSTRUCTION

(*Contributed by P. R. Strong*)

CHAPTER I

THE GENERAL PRINCIPLES OF FIRE RESISTANCE

There is no need to dwell upon the devastations of fire and the vast loss of life and property occasioned by it year by year.

Fire loss is absolute and irredeemable, no matter whether the price be paid by owners or by insurance companies. However fully property may be insured, it is rarely that the owner will escape without loss; while the occupier, even if he escape with his life, will in all probability suffer severely from depreciation of business.

Insurance companies profit considerably from sound fire-resisting construction, and it is not to be expected that owners will lay out large sums of money in avoiding fire risks if they are not met by corresponding reductions in their fire premiums. Fire insurance companies can do much, and are doing a great deal, to encourage satisfactory construction. The Fire Offices' Committee publish rules, compliance with which will be rewarded by reduced premiums. Still more might be done in the encouragement of the adoption of fire-resisting principles in order to obtain partial protection in the case of the smaller buildings.

The Degree of Resistance to Fire.—This may vary from *nil* in the very dangerous temporary wooden erection with flimsy hangings, to the almost complete fire resistance of the brick kiln or smelting furnace. Economy and practicability will warrant the complete adoption of fire-resisting principles in only a few instances, while a compromise must generally be arrived at between economy, practical convenience, and thorough fire-resisting principles.

In the following matter it will be attempted only to point out general principles and methods of construction, and these should be adopted as far as the needs of the case will allow.

The British Fire Prevention Committee has adopted three standards, namely, "Temporary Protection," "Partial Protection," and "Full Protection," according to the schedule given below. The International Fire Prevention Congress held in London in 1903 passed the following resolutions:—

"*Re the term 'Fireproof.'*—The Congress having given their consideration to the constant misuse of the term 'fireproof,' and its indiscriminate and unsuitable application to many building materials and systems in use, have come to the conclusion that the avoidance of this term in the general business and technical vocabulary is essential.

"*Re the term 'Fire-resisting.'*—The Congress considers the term 'fire-resisting' more applicable for general use, and that it more correctly describes the varying qualities of the different materials and systems of construction intended to resist the effect of fire for shorter or longer periods, at high or low temperatures, as the case may be; and it advocates the general adoption of this term in the place of the term 'fireproof.'

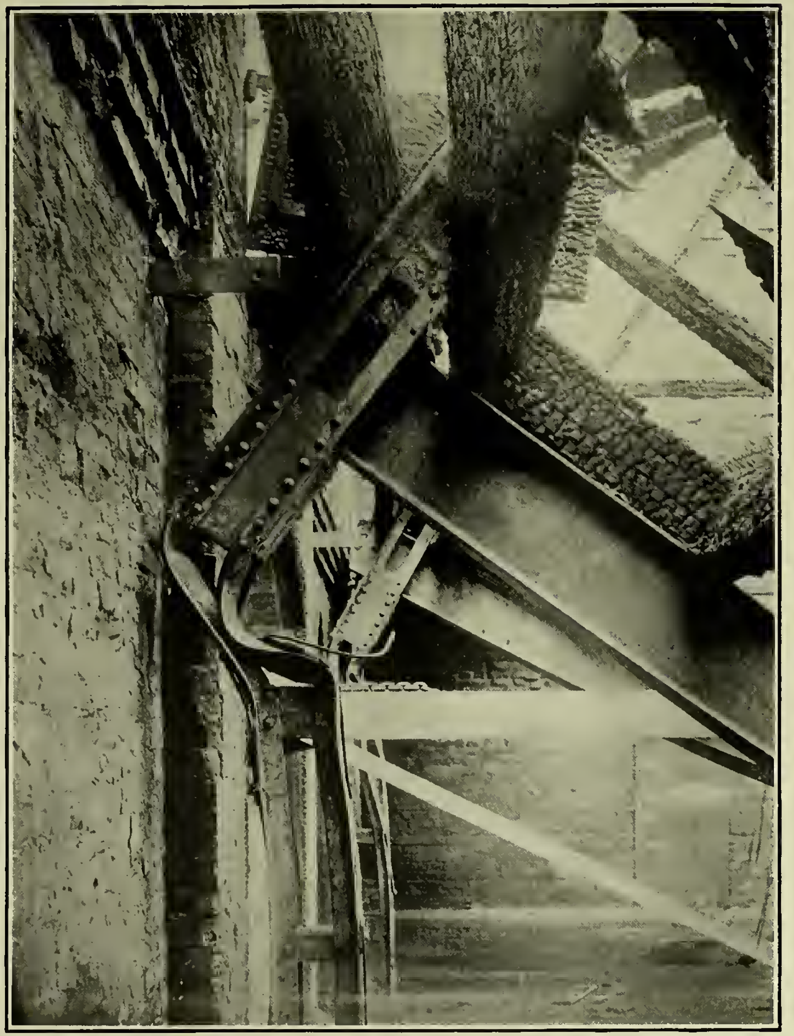
"*Re Standards of Fire Resistance.*—The Congress confirms the British Fire Prevention Committee's proposed standards of fire resistance, and hereby resolves that the universal standards of fire resistance shall in future be—

(1) Temporary protection;
(2) Partial protection;
(3) Full protection;

in accordance with the Committee's schedule."

"The exact and definite limit of these three classes is based on the experience obtained from numerous investigations and tests, combined with the experience obtained from actual fires, and after due consideration of the limitations of building practice and the question of cost."

The Executive of the Committee point out that the above standards can be popularly summarised as follows:—(*a*) That temporary protection implies resist-



Stanchion Crippled by Fire.

Stanchion Collapsed by Fire.

EFFECT OF FIRE ON UNPROTECTED STEELWORK AT THE GREAT HAMBURG FIRE OF 1894.

ance against fire for at least three-quarters of an hour. (*b*) That partial protection implies resistance against a fierce fire for at least one hour and a half. (*c*) That full protection implies resistance against a fierce fire for at least two hours and a half.

Incombustible and Fire-resisting Materials.—It is important for those unacquainted with the matter to clearly differentiate between incombustible and fire-resisting materials. For instance, steel or stone do not support combustion; but the first, if exposed to fire, will become useless for load-supporting purposes, as may be seen in Plate XI., which shows the effect of the great Hamburg fire of 1894 upon unprotected steel stanchions, while the latter will crack, spall or disintegrate to its complete destruction. Nothing, in fact, can be said to be *absolutely* fire resisting, and a selection of materials must be made from the most fire resisting of those which are suitable for the purpose.

Standard Table for Fire-resisting Floors and Ceilings.

Classification.	Sub-class.	Duration of Test at Least.	Minimum Temperature.	Load per Superficial Foot distributed.	Minimum Superficial Area under Test.	Minimum Time for Application of Water under Pressure.
Temporary Protection	Class A	45 mins.	1500° F.	Optional.	100 sq. ft.	2 mins.
	Class B	60 mins.	1500° F.	Optional.	200 sq. ft.	2 mins.
Partial Protection	Class A	90 mins.	1800° F.	1 cwt.	100 sq. ft.	2 mins.
	Class B	120 mins.	1800° F.	1½ cwt.	200 sq. ft.	2 mins.
Full Protection	Class A	150 mins.	1800° F.	2 cwts.	100 sq. ft.	2 mins.
	Class B	240 mins.	1800° F.	2½ cwts.	200 sq. ft.	5 mins.

Standard Table for Fire-resisting Partitions.

Classification.	Sub-class.	Duration of Test at Least.	Minimum Temperature.	Thickness of Material.	Minimum Superficial Area under Test.	Minimum Time for Application of Water under Pressure.
Temporary Protection	Class A	45 mins.	500° F.	2 in. and under.	80 sq. ft.	2 mins.
	Class B	60 mins.	1500° F.	Optional.	80 sq. ft.	2 mins.
Partial Protection	Class A	90 mins.	1800° F.	2½ in. and under.	80 sq. ft.	2 mins.
	Class B	120 mins.	1800° F.	Optional.	80 sq. ft.	2 mins.
Full Protection	Class A	150 mins.	1800° F.	2½ in. and under.	80 sq. ft.	2 mins.
	Class B	240 mins.	1800° F.	Optional.	80 sq. ft.	5 mins.

Standard Table for Fire-resisting Single Doors, with or without Frames.

Classification.	Sub-class.	Duration of Test at Least.	Minimum Temperature.	Thickness of Material.	Minimum Superficial Area under Test.	Minimum Time for Application of Water under Pressure.
Temporary Protection	Class A	45 mins.	1500° F.	2 in. and under.	20 sq. ft.	2 mins.
	Class B	60 mins.	1500° F.	Optional.	20 sq. ft.	2 mins.
Partial Protection	Class A	90 mins.	1800° F.	2½ in. and under.	20 sq. ft.	2 mins.
	Class B	120 mins.	1800° F.	Optional.	20 sq. ft.	2 mins.
Full Protection	Class A	150 mins.	1800° F.	2½ in. and under.	25 sq. ft.	2 mins.
	Class B	240 mins.	1800° F.	Optional.	25 sq. ft.	5 mins.

Objects of Fire-resisting Constructions.—A building may be erected entirely of the best fire-resistants known, yet bad planning may render the same building little better than a furnace in which its contents may be burnt and become a source of danger to all surrounding property.

The most important principles concerning resistance to fire are—

1st. Fire must be prevented from spreading to other people's property.

2nd. There must be means of escape for all persons in the building, no matter in what quarter the fire originates.

3rd. The rate of progress of fire must be sufficiently slow to allow time for all persons to make their escape.

These are matters which concern the public safety, and therefore must be controlled as far as possible by public regulations. The burning out of the contents of a building is a matter which chiefly concerns the owner, occupier, or insurance company, and from their point of view, at any rate, protection in this direction should be carefully studied.

The Restriction of Fire.—It would certainly be possible to erect a more or less habitable building in which no inflammable materials were used in its construction; but although furniture and other necessities inside a building may be rendered non-inflammable by chemical treatment, it is not to be supposed for a moment that the ordinary occupier of a dwelling-house will go as far as this to guard against fire. It is therefore the designer's duty to assume that fire may occur in any part of a building, and to so arrange the construction as to prevent its spread to other parts; but even the best efforts of the designer may be in part nullified by a thoughtless occupier.

If a room be surrounded by fire-resisting divisions a possible fire may burn itself out in this room without affecting other parts of the building; but if an opening be made in any of these divisions the fire will quickly spread to other parts of the building, and at the same time the opening will allow fresh supplies of air to reach the fire, which will consequently burn more furiously. For practical reasons there must of necessity be openings for doors, etc.; but nevertheless it is possible in this way to limit the fire to a single room, or at any rate to restrict its area until assistance can be obtained.

Thus it follows that proper fire-resisting construction largely consists in dividing a building horizontally and vertically, by means of fire-resisting floors, walls, and partitions, into a number of compartments or "risks."

The Protection of Structural Metal.—It is absolutely necessary at the outset to thoroughly realise that iron or steel, although incombustible, are *not* fire-resisting materials, and that wherever these materials are used to support loads they *must* be efficiently protected from fire. This precaution must be universal, no matter whether the building be partially or fully protected against fire. Considering the often serious consequences of neglecting this precaution, as evidenced by the result of the Hamburg fire (see Plate XI.), it is astonishing that a large amount of unprotected load-carrying ironwork should still exist in buildings.

Steelwork will fail, buckle, or bend under its load when raised to a light red heat. Cast iron is perhaps rather more reliable, but it is apt to break in two under the influence of fire, especially on the application of water.

The protection of wall-supporting girders is considered in Chapter XVII. Part II., while that of floor girders and joists will be treated of shortly.

The Protection of Stanchions.—This is of primary importance, for frequently the stability of a whole building depends upon stanchions, while, on account of the nature of their loading, they are particularly liable to buckle when affected by fire. Again, stanchions,

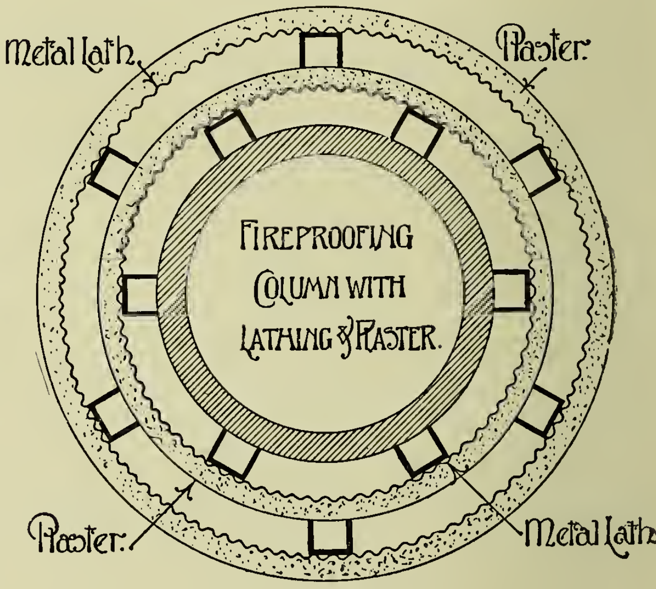


Fig. 205.

which stand out freely inside a room are particularly exposed to the attacks of fire; yet, notwithstanding these facts they have often been left unprotected while the protection of floor joists has received careful attention.

It must be remembered that all metal protection and all fire-resisting divisions must be capable of resisting not only the effects of fire, but also the erosive and quenching action of the water thrown by fire-engines.

Columns and stanchions are very often protected by winding metal lathing closely round them, on which plaster is laid. This is quite inadequate. Plaster will nearly always fall when exposed to fire for a short time, while the force of water from a fire-engine is certain to wash some of it away. If plaster is to be used for this purpose it must be applied in two thicknesses separated by an air space, as shown in Fig. 205; but this form of protection is not to be recommended.

Fig. 206 shows a hollow terra-cotta stanchion pro-

tection, a form which has been much used in America, the blocks being usually bound round with copper wire.

Terra-cotta varies in texture from a hard dense form

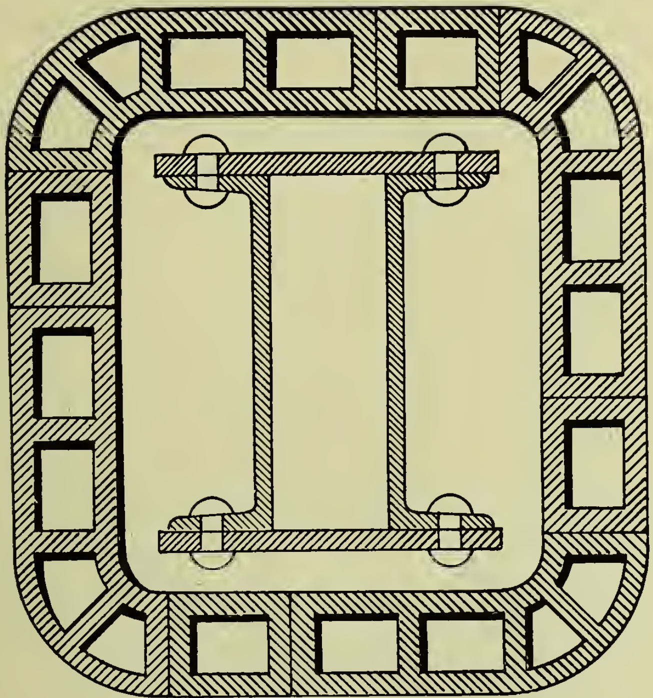


Fig. 206.

to a porous quality, the latter being made by mixing the clay with chaff, sawdust, etc., which is consumed when the object is burnt, leaving the terra-cotta in a porous

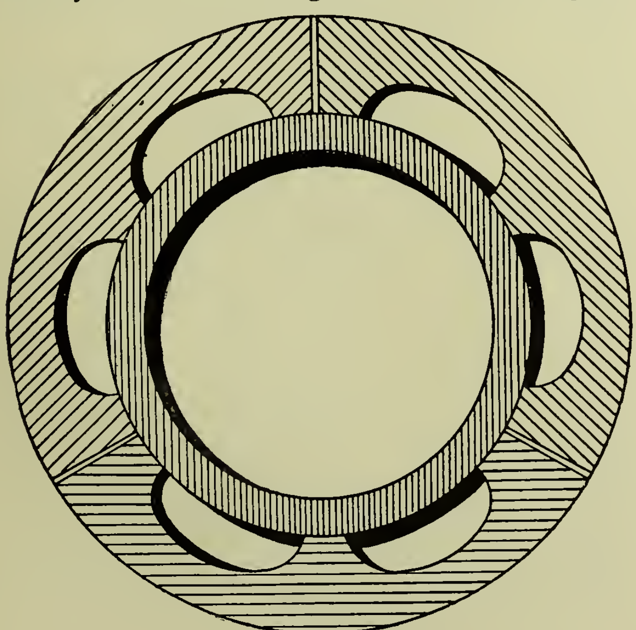


Fig. 207.

condition. The hard burnt variety conducts heat considerably more quickly than the porous, and the latter is distinctly preferable.

Protection of the form illustrated in Fig. 206 has, in the majority of cases, saved the stanchion that it has been applied to, but has become a total loss in itself. The outer face becomes detached, the internal webs cracking from unequal expansion. This effect is produced with both hard-burnt and porous terra-cotta, the former probably being the worse in this respect. The water quenching probably accounts for a good deal of the damage done. The defect just mentioned occurs with all the forms of hollow tile protection commonly used, in which the faces and webs are formed with a minimum thickness of material. It is, however, by

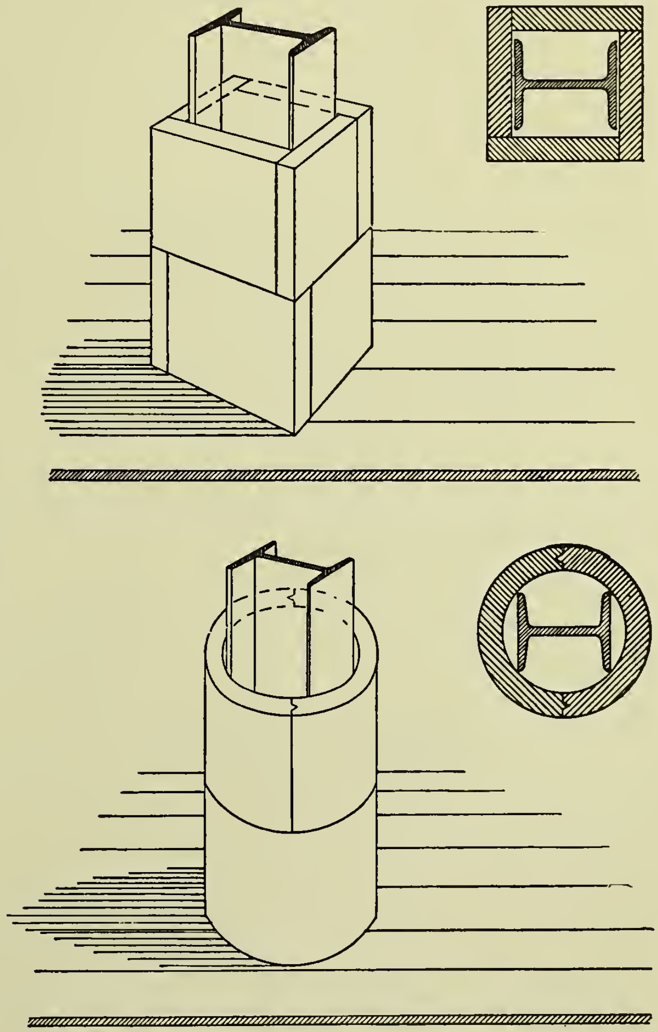


Fig. 208.

no means certain that this form cannot be satisfactorily used, and it is probable that "tiles" made of porous terra-cotta with sufficient webs, no part being less than $1\frac{1}{2}$ inch in thickness, and with all angles rounded, will be highly satisfactory. There is no doubt as to the efficiency of solid blocks of porous terra-cotta, and Fig. 207 shows a cast-iron column protection of this nature. Fig. 208 illustrates the protection of stanchions with slabs formed of Jabez Thompson's "Terrawode Brickwood."

The space between the terra-cotta and metal should

always be filled in with concrete, for if this is not done a flue may be formed throughout the whole height of the stanchion, when, on a tile being displaced, fire may be led in through the hole thus formed, spreading the fire and causing the collapse of the stanchion.

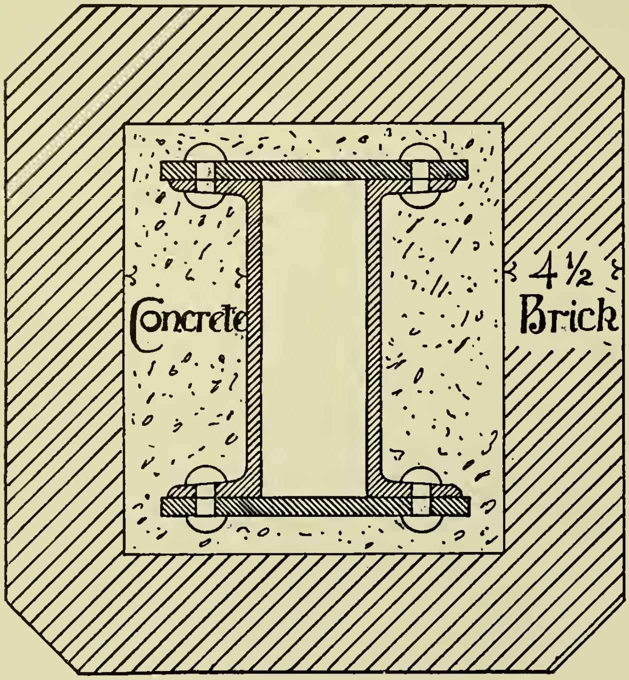


FIG. 209.

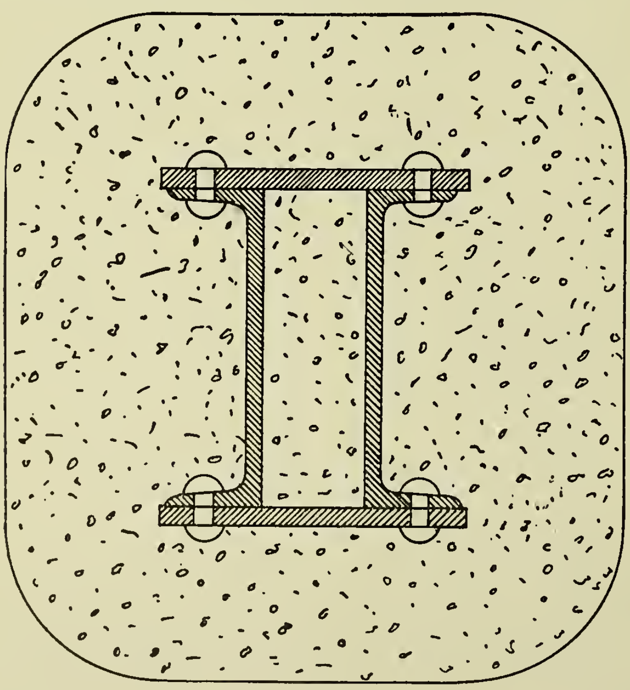


FIG. 210.

An efficient protection may be made with brickwork, as shown in Fig. 209, the space between brick and metal being filled in with cement concrete.

A very good protection may be formed of concrete alone, as shown in Fig. 210, in which the stanchion is also shown filled with concrete for the sake of protection from corrosion. For the latter reason, cement concrete is highly to be recommended for protecting all steel work from fire; while the quality of various aggregates, as regards fire-resisting properties, is mentioned on page 171. It will be noticed that the corners of the concrete are shown rounded. This is desirable, not only to resist blows, but also because concrete is liable to spall at angles, and a rounding of 3 inches radius or more will lessen the risk of this; it is well, in fact, to round the corners of any form of protection. If it is desired to give the concrete a circular section, hinged moulds, as shown in Fig. 211, may be used. From its

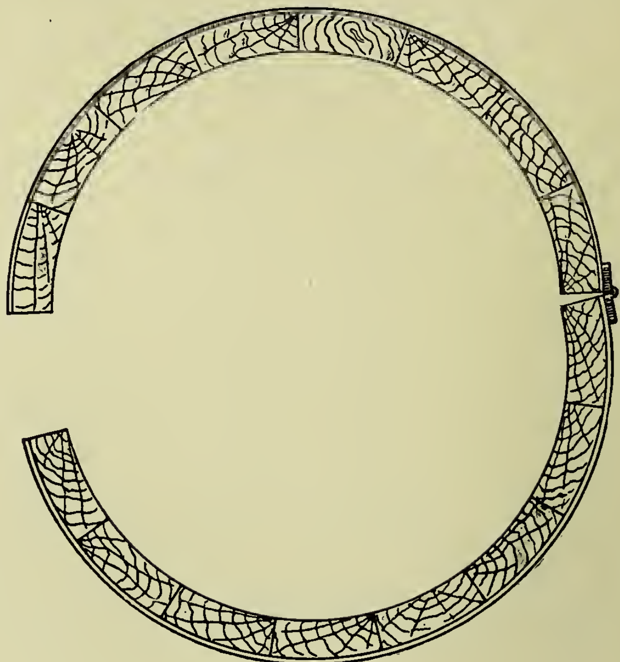


FIG. 211.

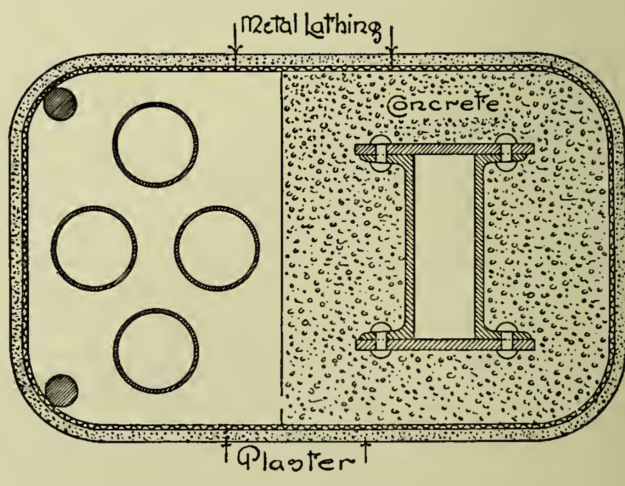


FIG. 212.

complete absence of corners, circular protection is to be recommended for the reason just stated.

The thickness of protection, which should preferably be not less than 4 inches, must not be decreased or cut away for any purpose whatever. There have been many cases where the tile protection has been cut and split the whole length of the stanchion, to allow pipes to be led up by its side. If it is necessary to carry up pipes by the side of stanchions, they must be outside the full thickness of protection, and this may be done as shown in Fig. 212. Again, no wooden fixing strips must be built into any form of protection, for this has often been a cause of failure.

Where a stanchion is exposed to blows, the fire-resisting protection must be shielded, unless it be of a very solid description.

CHAPTER II

FIRE-RESISTING CONSTRUCTION: HORIZONTAL DIVISIONS

Floors.—There are many patent fire-resisting floors on the market, and several are illustrated later in this chapter.

The selection of the type of floor to be used must depend upon—

Resistance to fire and water.
Strength and rigidity.
Moderate cost.
Moderate weight.
Simplicity of construction.

Most of the various patent floors in which the steelwork is properly protected will satisfactorily resist the effects of fire; however, unless founded upon proven principles, the full resistance of a floor should not be taken for granted before official tests have been made upon it. The British Fire Prevention Committee have a testing station at which the manufacturer, at his own expense, may have his specialities officially tested and reported upon.

Cement Concrete is now almost exclusively used in the construction of fire-resisting floors, while terra-cotta is employed to some extent in the shape of a permanent centering. The advantages of concrete as a fire-resisting material are its low heat-conducting power, and its small expansion under heat. The efficiency of concrete was for some time under considerable doubt, owing to the effect of fire upon cement briquettes, causing them to lose a very large percentage of their strength. It was then concluded that concrete would similarly lose its strength, but this conclusion is only to a slight degree justified. The exposed surface of concrete becomes dehydrated up to a depth of about one inch under severe conditions, or even more with poor concrete. This layer, which may be washed off on the application of water, protects the underlying concrete to some extent. Under ordinary conditions the effect, as a rule, would hardly be as deep as is indicated above, especially when the surface is protected with plaster.

The fire-resisting properties of cement concrete depend upon its quality, as indeed do those of all materials. But, apart from the care taken in mixing, and the quality of the cement, the nature of the aggregate used largely affects its efficiency. Tests apparently point to the conclusion that porous and elastic materials, on account of their low conduction of heat, make the best aggregates.

The aggregate which has given distinctly the best results, contrary to what might perhaps have been expected, is coke-breeze. This material has the further advantages that it is light and cheap, and also that nails can be driven into it, and that it can be cut, while the chief objection to its use lies in its liability to cause the corrosion of steel. Another objection put forward is the great variation in the quality of coke-breeze, very poor material of doubtful strength and fire-resisting qualities being largely used; and many for these reasons avoid it altogether.

Concrete made with broken brick of good quality is also an excellent fire-resister, while it is light and has not the objection pertaining to coke-breeze in causing the corrosion of steel.

Furnace slag is a good aggregate, but it is heavy.

Concrete made with broken stone as an aggregate is heavier than that formed with coke-breeze or brick, but generally offers satisfactory resistance to fire, although this must largely depend upon the stone used. Thames ballast and similar aggregates are particularly to be avoided.

Particulars of tests carried out by the British Fire Prevention Committee on slabs of concrete, in order to ascertain the quality of various aggregates, are given below, being taken from their report. It may be noted that the slabs had not the protection of plaster rendering, and that the quality of the cement used was particularly good.

Object of Test

To record the effect of a fire of 3 hours' duration, the temperature to reach 1800° Fahr., but not to exceed 2200° Fahr., followed by the application of water for two minutes.

Note.—The area of the floor under investigation was to be divided into seven equal bays of different aggregates, the quantity and quality of Portland cement used being identical for each bay, and the nature of the concrete used being as follows:—

No.			Parts by Volume.
1.	Slag concrete	Blast furnace slag	3
		Clean sand	2
		Cement	1
2.	Broken brick concrete	Broken brick	3
		Clean sand	2
		Cement	1
3.	Granite concrete	Broken granite	3
		Clean sand	2
		Cement	1

No.		Parts by Volume.
4. Burnt ballast concrete	Burnt ballast	5
	Cement	1
5. Coke-breeze concrete	Coke-breeze	5
	Cement	1
6. Clinker concrete	Furnace clinker	3
	Clean sand	2
	Cement	1
7. Thames ballast concrete	Thames ballast	3
	Clean sand	2
	Cement	1

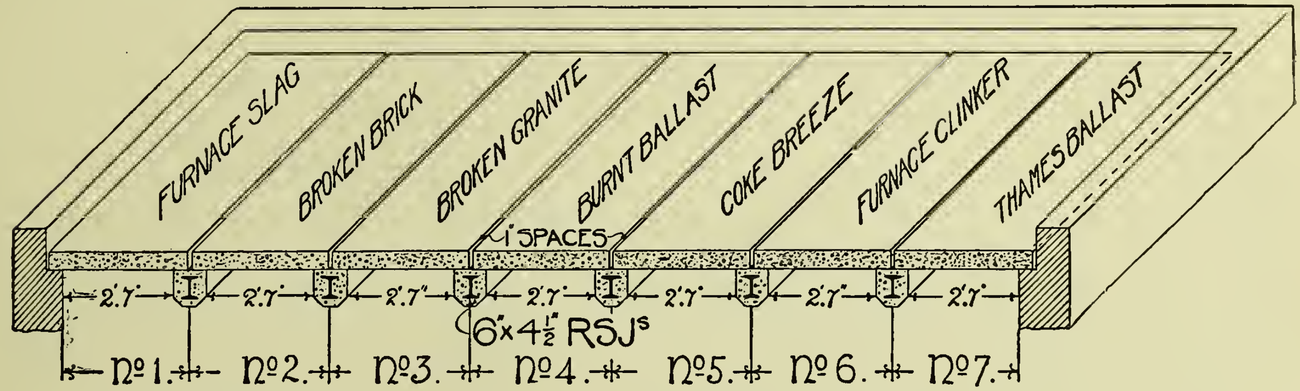
On the application of water more plaster was washed off the beams than had fallen during the fire test, and some of the concrete from the under side of bays Nos. 3, 4, 5, 6, and 7 was washed off.

All the slabs remained in position.

Bay No. 6 was flat on the soffit, all the others being convex on the under side, No. 7 (the worst) to the extent of 1½ inch.

On the removal of the load it was found that bays Nos. 1, 2, 3, 6, and 7 were cracked across, No. 7 being worst. (See Fig. 213.)

Terra-cotta.—Remarks made in reference to stanchion



Summary of Effect

No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 7.
Top: Slab cracked across in two places.	Top: Slab cracked across in three places; slight curve downwards.	Top: Slab cracked across in three places; curved downwards about ½ inch.	Top: No cracks; not curved downwards.	Top: No cracks; not curved downwards.	Top: Slab cracked across in two places; curved downwards about ⅜ inch.	Top: Slab cracked across in very many places; curved downwards about 2 inches.
Under side: Curved downwards ¼ inch; slight cracks visible.	Under side: Curved downwards ¼ inch; slight cracks visible.	Under side: Curved downwards ½ inch; no cracks visible; about 1 inch washed off by water.	Under side: Not curved downwards; no cracks visible; about 3 inches washed off under side (in parts) by water.	Under side: Not curved; no cracks visible; about 1 inch washed off under side (in parts) by water.	Under side: Not curved; one slight crack visible; pitted in places about 1 inch deep by water.	Under side: Curved downwards 1½ inch, and bad cracks all over in all directions, mainly longitudinally; much washed off by water.

Fig. 213.

The total area of the floor under investigation was to be at least 200 feet super.

The soffit of each bay exposed was to be about 10 feet by 2 feet 7 inches, the thickness being 5½ inches.

The floor was to be loaded with 224 lbs. per foot super.

The centering was to be struck 14 days after completion of the floor. The time allowed for drying was 40 days (Autumn).

Summary of Effect.

In ten minutes after the gas was lighted the plaster began to fall off the beams, and it continued to do so until the end of the test.

Towards the conclusion it was observed from the top of the hut that the edges of bays Nos. 1, 6, and 7 were red hot, No. 7 being the worst.

protection apply here equally. Terra-cotta has been largely used in America for the construction of floors, in the form of flat arches built up of hollow blocks between the joists. Many fires have shown that this method is hardly trustworthy, for the exposed faces of the blocks break away, as in the case of stanchion protection. Terra-cotta, when used, should be of the porous kind, well supplied with webs, and having thick walls.

It may be mentioned here that, where an arched floor is intended to carry its load as an arch, tie rods must be used at intervals between every pair of beams, and this tie rod must be efficiently protected against fire.

Suspended Ceilings.—Some systems of floor construction attempt to protect the ironwork by means of plaster on suspended metal lathing, and, as mentioned in connection with stanchions, protection of this kind is

not reliable. Floors constructed on this principle may very probably prevent the spread of fire, but the necessity for their reconstruction after a conflagration is more than likely.

Again, other systems attach the protection by means of metal clips, but this system is of doubtful value unless the clips are well out of the reach of the attacks of fire.

If the metalwork is properly protected, hollow floors are desirable for the sake of lessening the transference of sound, as well as to lighten the floor.

A few of the fire-resisting floors upon the market will now be touched upon. It is impossible to compare the relative merits of various floors until prices are known, and this question of course cannot be discussed here. A great advantage in employing special firms to carry out flooring is that it is done by men who are thoroughly

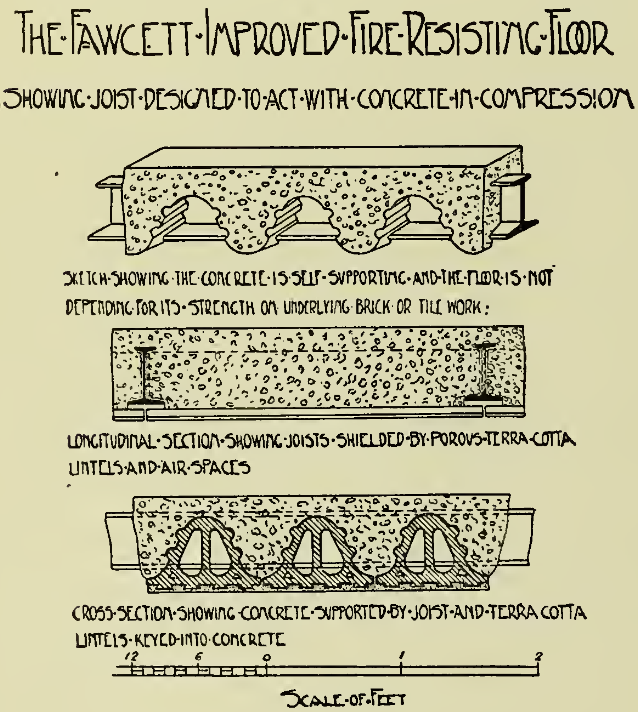


FIG. 214.

used to this form of work; and this results in the work being well and quickly done, while the cost will probably not be more than if carried out without the help of specialists.

The Fawcett Improved Fire-resisting Floor.—This is illustrated in Fig. 214. It consists of a concrete floor with a permanent terra-cotta centering. The concrete constitutes the strength of the floor, the terra-cotta lintels being intended to act merely as centering, and as a protection to the lower flange of the joists, at the same time serving to lighten the floor and to present a suitable surface to receive plaster. The lintels, which are 2 feet 6 inches long, are made of porous terra-cotta, and are well provided with webs.

It is pointed out by the makers that these tubes may be connected to the outer atmosphere, which will allow of the circulation of air round the lower flanges of the joists, tending to keep them cool in case of fire. The advantage of this is doubtful when considering the preservation of the steel against rust; while if, in case of fire, a portion of the terra-cotta be broken away, the air space may act as a flue, and actually draw the fire in and along the surface of the metal.

The use of a special joist is also shown in this illustration, it being intended that the concrete round the upper flange shall help to resist the compressional stress.

Homan's Fire-clay Hollow Brick Floor.—This is illustrated in Fig. 215, and is similar in principle to the floor just considered, except that the lintels are laid

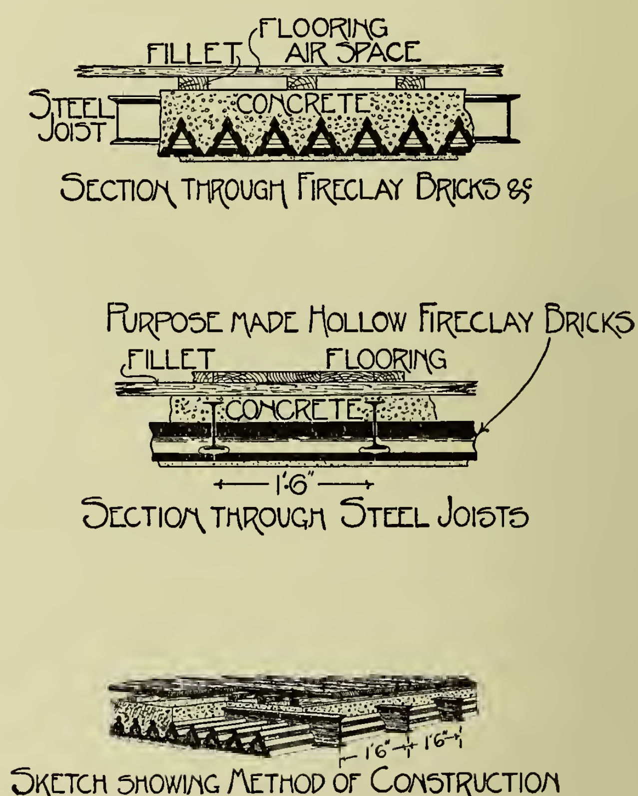


FIG. 215.

directly across between the joists instead of diagonally.

Dawnay's Patent Solid Fireproof Tile Floor (Fig. 216).—This floor, like the above, employs a terra-cotta tile as a permanent centering, but in this case the lintels are solid.

Floors of this description have the objection that irregularly shaped portions of floor cannot easily be filled in, while their doubtful point is as to whether the terra-cotta may not become broken away from the under side of the joist, thus leaving the metal exposed to fire. Official tests on the latter point would be of great service. The tiles, at any rate, serve to protect the surface of the concrete; for porous terra-cotta, if of solid or suitable proportions, is probably the most effective fire-resistant known.

Potter & Co.'s "A" Floor (Fig. 217).—In this floor permanent centering for the concrete is supplied

by corrugated iron, while the lower flange of the joist

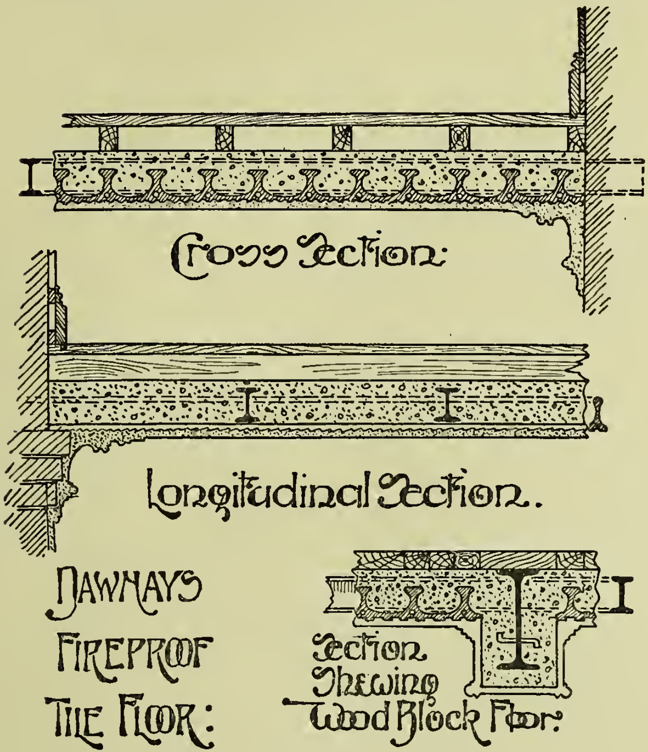


Fig. 216.

is doubly protected by a fire-clay shoe and a suspended ceiling. It may be observed that in floors of this kind, either the thickness of concrete at the centre between joists must be sufficient to resist the load as a beam, or, if the concrete is to act as an arch, it must exert a thrust, which must in turn be met by tie rods or by the resistance of walls and the rigidity of other parts of the floor.

The Columbian Floor, of which a general view is given in Fig. 218 and details in Figs. 219 and 220. This floor is formed of concrete, strengthened and stiffened by bars of the shape shown in Fig. 219, various sizes being used for different purposes; the small size illustrated is used in the construction of the flat ceiling. The bars used in the flooring are hung from the upper flange of the joists by metal clips or stirrup pieces of the form shown in Fig. 219. In the panelled construction the lower flanges of the joists are protected by special concrete slabs (Fig. 220), which are attached to the flanges by the metal clips shown. In the flat ceiling construction the concrete ceiling, $2\frac{1}{2}$ inches thick, containing 1-inch ribbed bars, is first formed upon centering. A centering is then placed upon the concrete ceiling, and on this the floor is formed. The latter centering is afterwards removed through an opening purposely left in the ceiling, which opening is afterwards filled with a slab of concrete of special shape, moulded to fit the opening. These openings may be seen in the bottom illustration of Fig. 218. There is no doubt that this form of floor is thoroughly strong

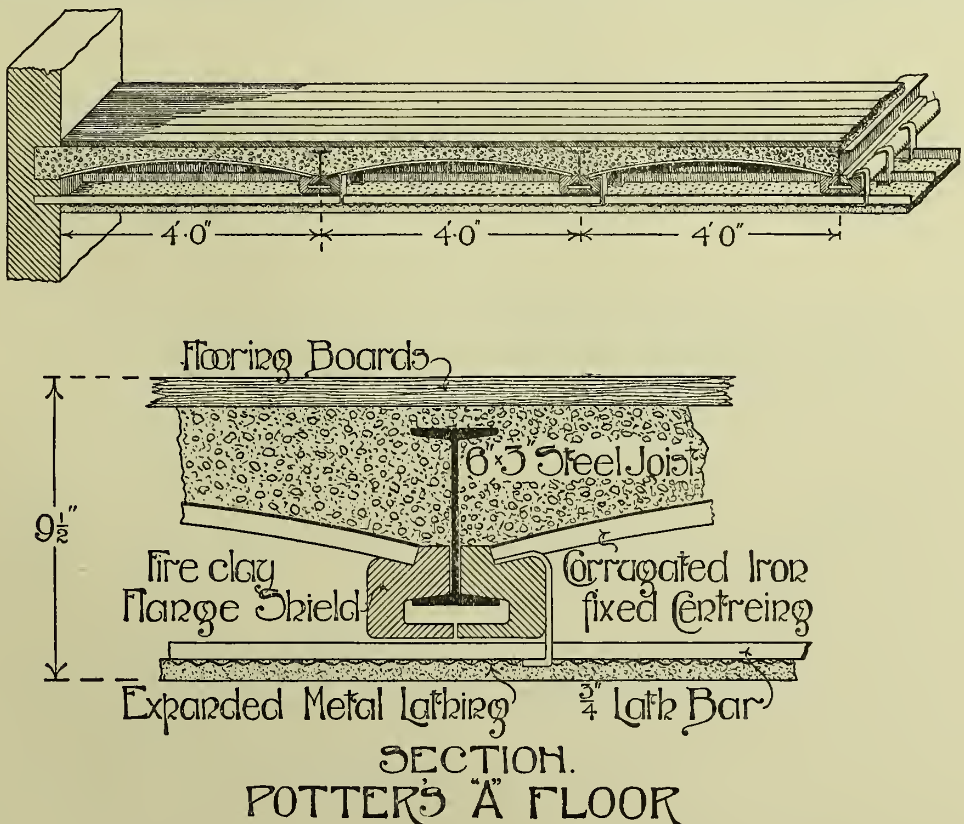


Fig. 217.

and fire-resisting, all the steelwork being well protected. It may here be mentioned that the use of special forms of members is often for no further purpose than to secure a patentable construction.

Homan's Improved Steeled Concrete Floor (Fig. 221).—In this floor steel rods, passing through the webs of the joists, are imbedded in the lower part of the concrete. These rods strengthen the concrete between the joists by resisting part of the tensional stress in its lower half, while it is claimed that they tie the bays securely together, resulting in less thrust upon the walls.

A concrete slab supported freely at either end may be insufficient to carry its load as a beam on account of its small resistance to tension; however, if the slab be rigidly confined at its ends it may act as an arch, as indicated in Fig. 222, exerting a thrust which is resisted by the rods embedded in the concrete.

Homan's Patent Concrete and Steel Floor (Fig. 223).—In this floor special tee-shaped members are embedded in the lower part of the concrete, in order to take the tensional stress, the vertical portion being given a curved section to ensure greater intimacy and adherence between the two.

A Floor by the Expanded Metal Company (Figs. 224 to 228).—In this floor the tensional stress in the concrete slab is met by "Expanded Metal."

Expanded metal is formed by cutting slits in sheet

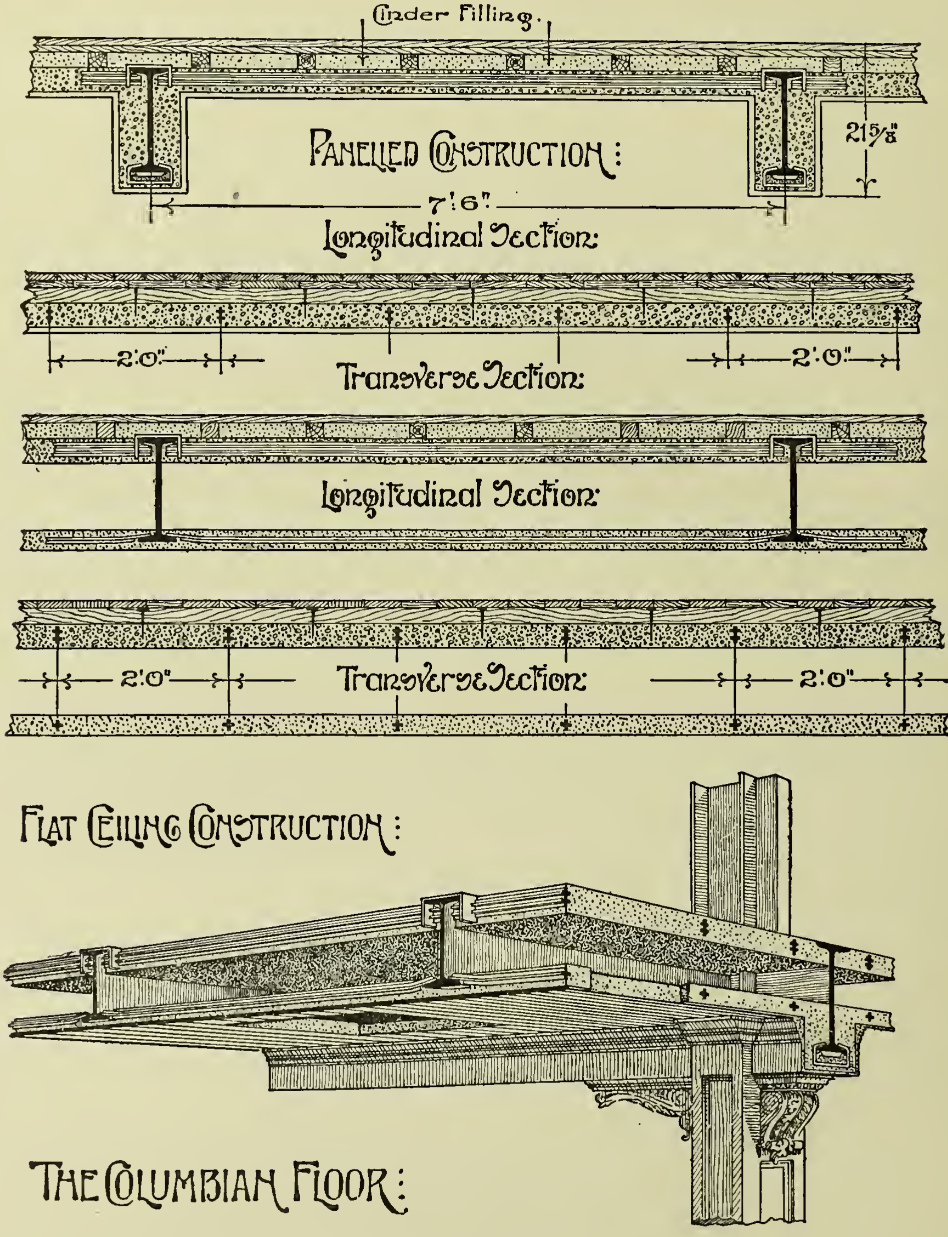


Fig. 218.

metal, which is then stretched and opened out to the form shown in Fig. 225. It may be supplied up to 16 feet width, measured the long way of the mesh, and of any convenient length the short way of the mesh. The material is described by the shortest dimension of

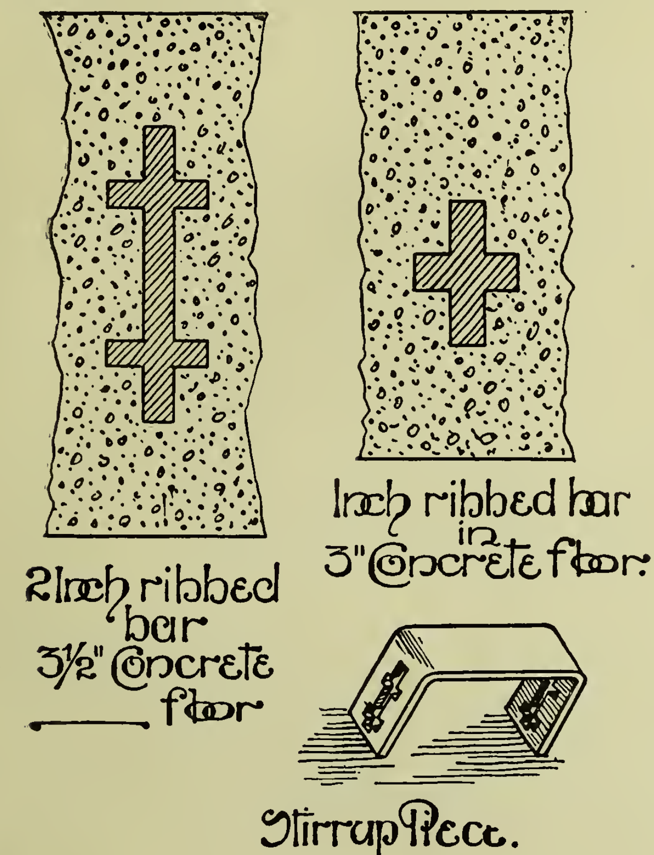


Fig. 219.

mesh, the various meshes being known as $\frac{1}{4}$, $\frac{3}{8}$, $\frac{3}{4}$, $1\frac{1}{2}$, 3 and 6 inches, while a number of different weights can be obtained in each size. The size of mesh generally used in reinforcing concrete is 3 inches, while $\frac{3}{8}$ inch is

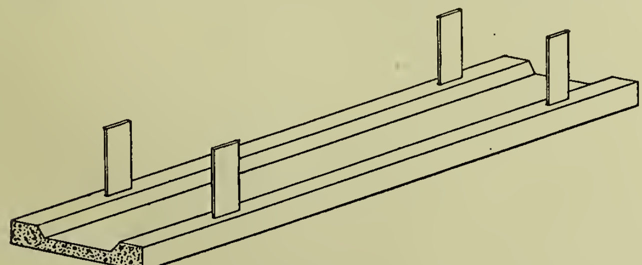


Fig. 220.

used as lathing. This material is useful for many purposes.

In forming the floor the expanded metal is laid upon the centering, with the edges of the adjacent sheets lapped over one another. The concrete is then spread over the surface and thoroughly tamped down (Fig. 226). This process results in the metal becoming embedded in the lower part of the slab, leaving a smooth surface underneath. The ceiling is suspended by bars

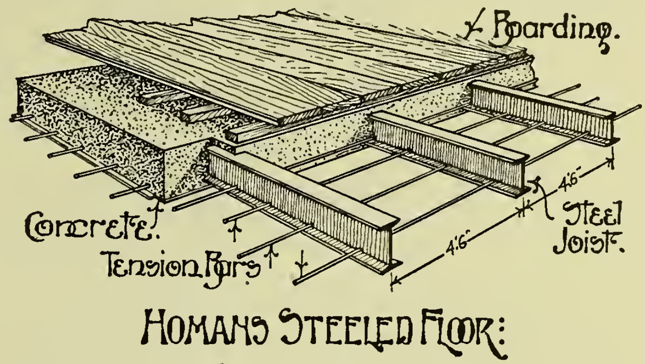


Fig. 221.

attached to the lower flanges of the joists with clips, as shown in Fig. 227, the expanded metal lathing being again clipped to these bars. The question as to the

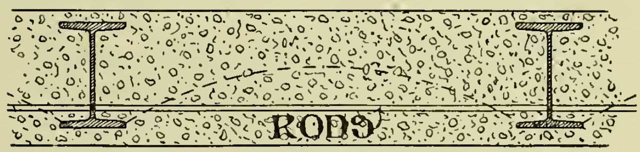


Fig. 222.

desirability of a suspended plaster ceiling as sole protection to the metal work has already been discussed.

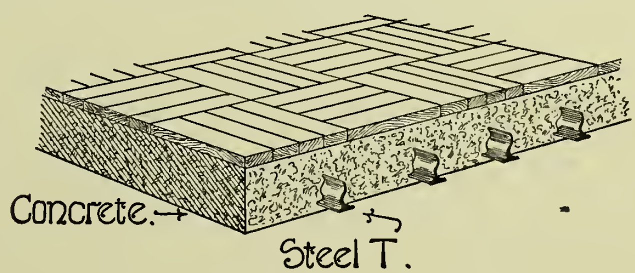


Fig. 223.

A section is shown in Fig. 228 by the same company, in which the joist is fully protected. It may be mentioned

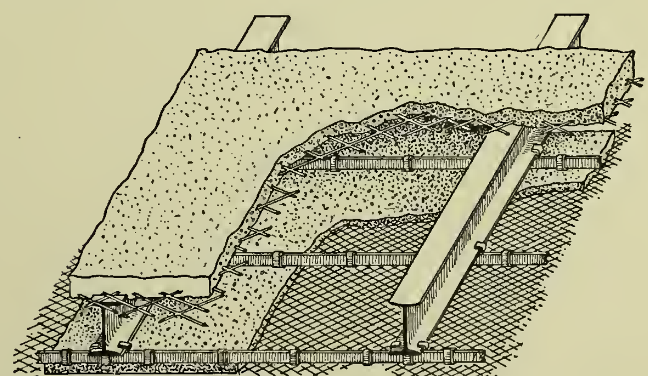


Fig. 224.

tioned that an even ceiling is not so liable to destruction as one with sharp projections upon it, and thus the

protection of projecting girders with nothing but plaster on metal lath is particularly objectionable.

Lindsay's Steel Trough Flooring (Fig. 230).—This form of flooring is particularly useful where heavy loads

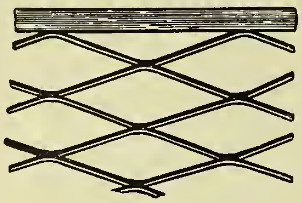


FIG. 225.

have to be dealt with. It can be used without other support for spans up to 50 feet. The under side may be protected with concrete blocks suspended as shown in the illustration.

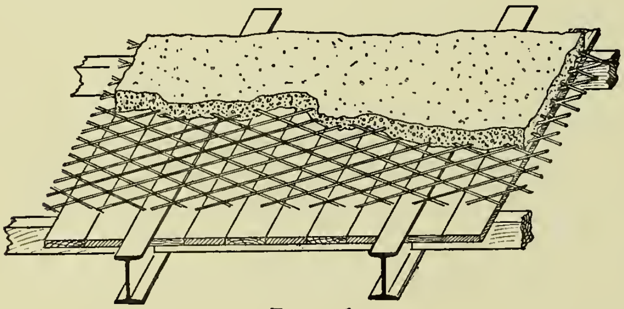


FIG. 226.

Pumice concrete, which is used by Messrs. Lindsay & Co., is exceptionally light, and of high fire-resisting quality. The chip concrete, also used by them, is com-

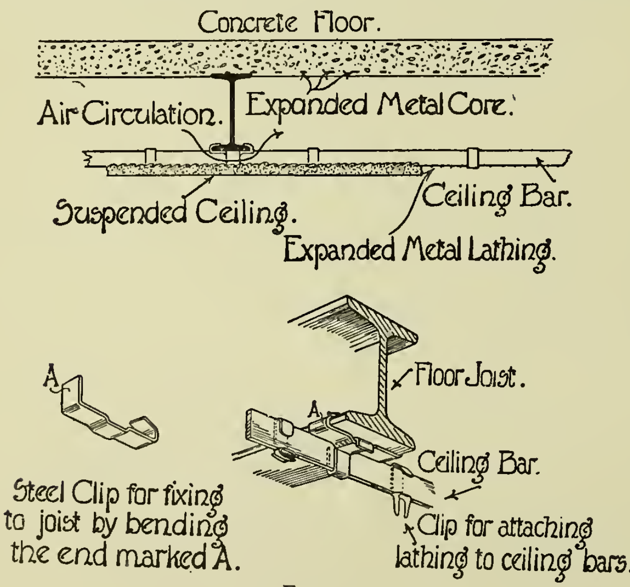


FIG. 227.

posed of crushed fire tile, and forms a very good fire-resistant.

Fig. 229 shows a method of forming the protection to floor girders by the same company.

When a large girder rests upon a masonry wall, expansion should be allowed for, otherwise on heating it may force the wall over.

Having constructed a satisfactory fire-resisting floor, it is important that no hole of any sort shall be left in it. Holes are frequently knocked in floors through which pipes may be passed, and more often than not these holes are left only partly filled by the pipe. It is

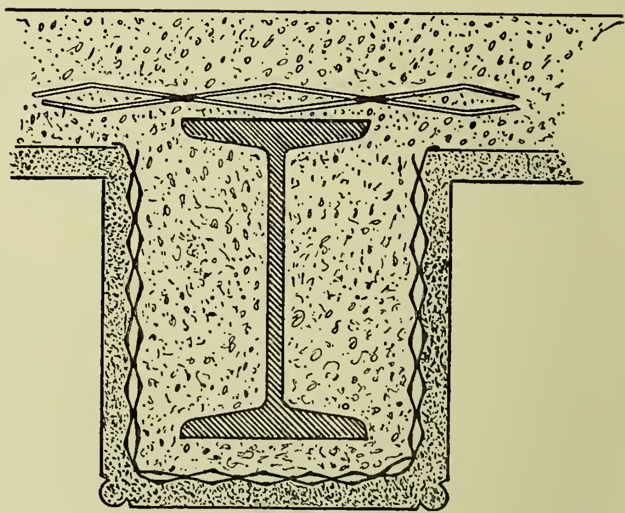


FIG. 228.

of the greatest importance that such holes should be carefully filled in with concrete or cement, and that the whole floor should form a complete barrier to the passage of fire.

ROOFS.—Under the heading of horizontal division, roofs must also find a place. A flat fire-resisting roof may be constructed in the same manner as the fire-

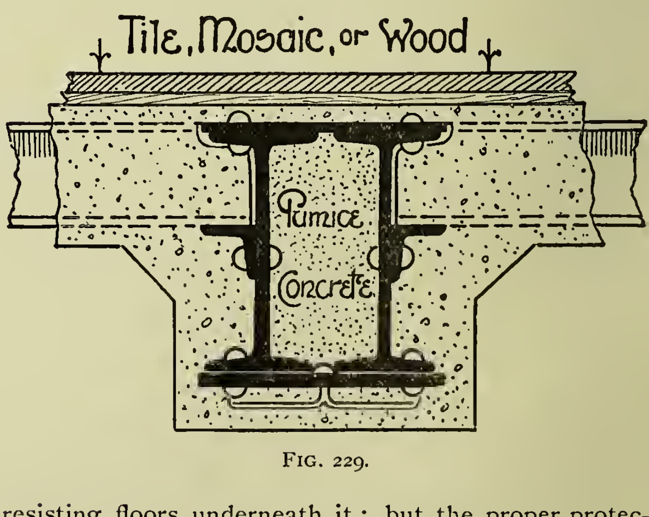


FIG. 229.

resisting floors underneath it; but the proper protection of a steel roof truss is not such a simple matter, and for this reason buildings in which fire-resisting principles have been carried out are often covered with a flat roof.

A Mansard roof is a very favourite finish to a fire-resisting building, while its protection is a comparatively simple matter. The steeply sloping sides of the roof

are built up in the same manner as are the partitions below it, while the low pitched portion is converted into a flat roof, constructed as an ordinary fire-resisting floor. The Ritz Hotel exemplifies this form of roof (see Plate VI.).

It is, of course, necessary that a flat roof should be given a certain amount of slope, and consequently a ceiling formed on its under side would likewise slope. If a horizontal ceiling be desired, this can readily be obtained by suspending it from the roof with hangers or clips of varying length; but a suspended plaster ceiling must not be depended upon to protect the steel-work of the roof. If the ceiling be made thoroughly fire-resisting, as in the case of the Columbian floor, this may be depended upon to protect the lower flange of the roof beams; but only so long as no opening or possible means of access to the space between roof and ceiling is provided; for, if access be permitted, the space, if sufficient, will soon be used as a deposit for lumber, which will form fuel for the destruction of the roof. If a sloping roof be employed, a parapet wall must be used to protect firemen from falling slates and tiles,—and this is now compulsory in London for high buildings.

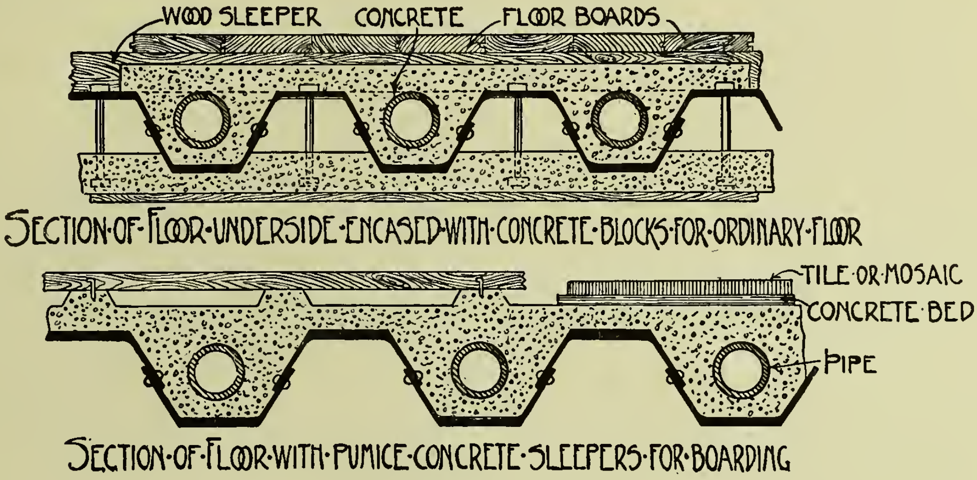


Fig. 230.

CHAPTER III

FIRE-RESISTING CONSTRUCTION: VERTICAL DIVISIONS

Party Walls.—Fires have constantly demonstrated the superiority of good brickwork of suitable thickness over all other forms of walling; and party walls, in which ornamentation can take no part, should always be constructed of this material. In towns the party walls form the only protection between adjacent houses, and they must consequently be made fully fire-resisting, and no opening or reduction of thickness may be allowed in any part. Building laws govern the thickness of this wall, and the thicknesses laid down by them will generally enable fire to be restricted in this direction.

The London Building Act specifies that the party walls of all buildings, except those of the warehouse class, are to be carried 15 inches above the roof, measured at right angles to the slope; while in the case of warehouses, etc. the height is to be 3 feet. This provision is obviously necessary to prevent the spread of fire from roof to roof.

External Walls.—The materials used in the construction of external walls consist of stone—granite, marble, limestone, and sandstone—as well as concrete, terra-cotta, and brick. Of these, good brickwork is distinctly preferable. Stone, although incombustible and a bad heat conductor, is seriously damaged by fire, and it is chiefly from the point of view of cost of reconstruction after fire that it is to be recommended that stone should not be used, while for load-carrying piers or columns its use should be more strictly avoided, for their effective section may be reduced to a serious extent by the action of fire. Taking the above-mentioned materials in order:—

Granite, when exposed to fire, will disintegrate and crumble away, while small pieces will fly off with slight explosions.

Marble and Limestone become calcined and "spall," —that is, small pieces break off.

Sandstone is generally better than other forms of stone from the fire-resisting point of view, but under a moderate fire it will spall considerably.

Concrete, if made of good cement and suitable aggregate, is an excellent fire-resistant, but if made of flint or Thames ballast is liable to disintegrate (see Chapter II.).

Terra-cotta has been much used for the fronts of buildings, both on account of its superior fire-resisting qualities, as well as from its adaptation to the repetition of mouldings and ornament. Recent fires, notably the great Baltimore fire (February 1904), have shown, however, that terra-cotta does not escape so well as was expected, particularly the more highly ornamented portions; but if it be tough, moderately porous, and of good thickness, well strengthened with webs, it is probable that even projecting mouldings would escape any considerable injury.

Brickwork of good quality, as previously stated, is eminently the most satisfactory material. It may crack and even spall in places, but the body of it will be unharmed; at the same time, a brick facing attached to the main wall with metal ties cannot be considered satisfactory, for it is liable to be stripped bodily off in case of fire.

Thus, if expensive reconstruction is to be avoided, the designer, for the facing of his walls, is apparently limited to brickwork or terra-cotta of solid proportions. Many will prefer to risk the chances of fire and consequent reconstruction for the benefit of the use of stone; but in this case stone should be employed only as a facing, the backing being of brickwork or concrete.

In the case of steel framed buildings, as erected in America, the external wall is often only a veneer, acting as a screen and as a protection to the steelwork, the protection, however, being often of very doubtful quality. The spandril section in Fig. 193 shows the sort of protection afforded to the wall girders, whereas in many cases it is of a still more doubtful character. Terra-cotta suspended in this way by iron hangers cannot be depended upon. To avoid the necessity for protection of the nature just referred to, the external walls should be brought up continuously from ground to roof, not necessarily self-supporting, but of sufficient thickness to carry an arch or other bridging over the window openings, which will in turn support protection to the wall girders in the form of a solid wall. In London the thickness of the walls specified by the Building Act usually makes the adoption of this arrangement a matter of course.

The fire-resisting external screen of a building is provided to little purpose if unlimited and unprotected openings be allowed in it in the shape of windows; but this will be considered later.

Internal Walls and Partitions.—Here again the most efficient division consists of a brick wall 9 inches or more in thickness, while a brick wall 4½ inches thick, if not of too large an area, will amply suffice in the majority of cases.

In order to save space and to reduce weight, many

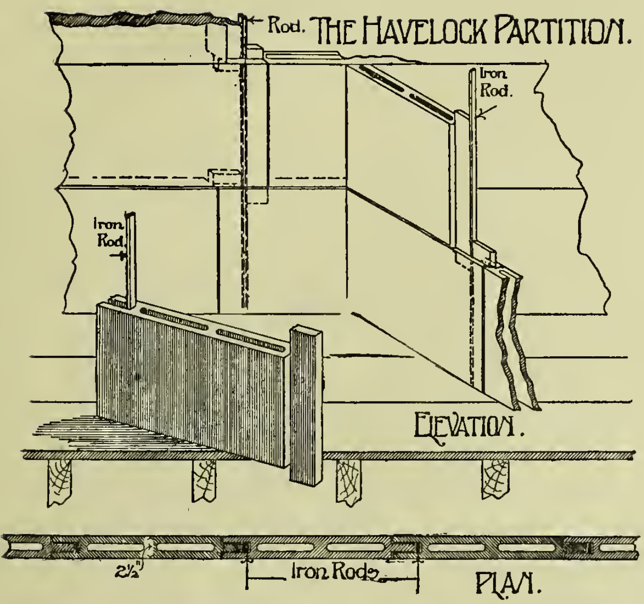


FIG. 231.

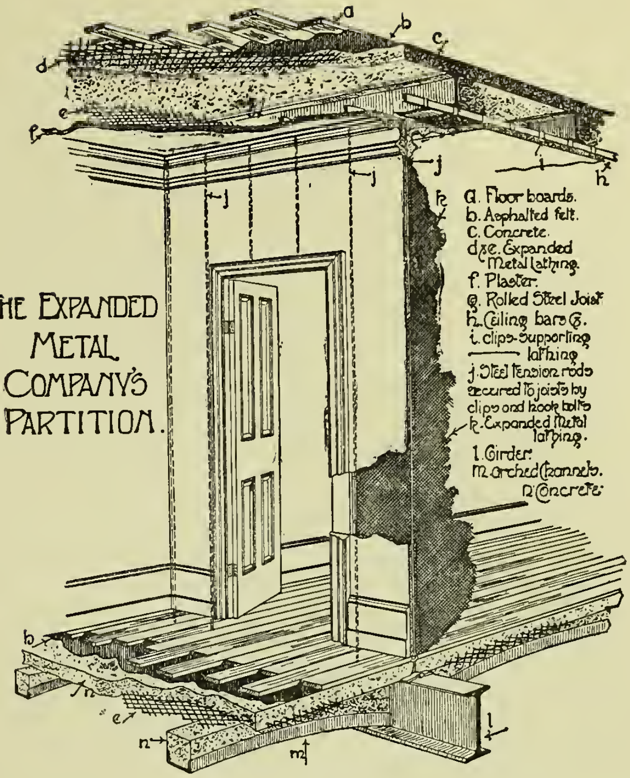


FIG. 232.

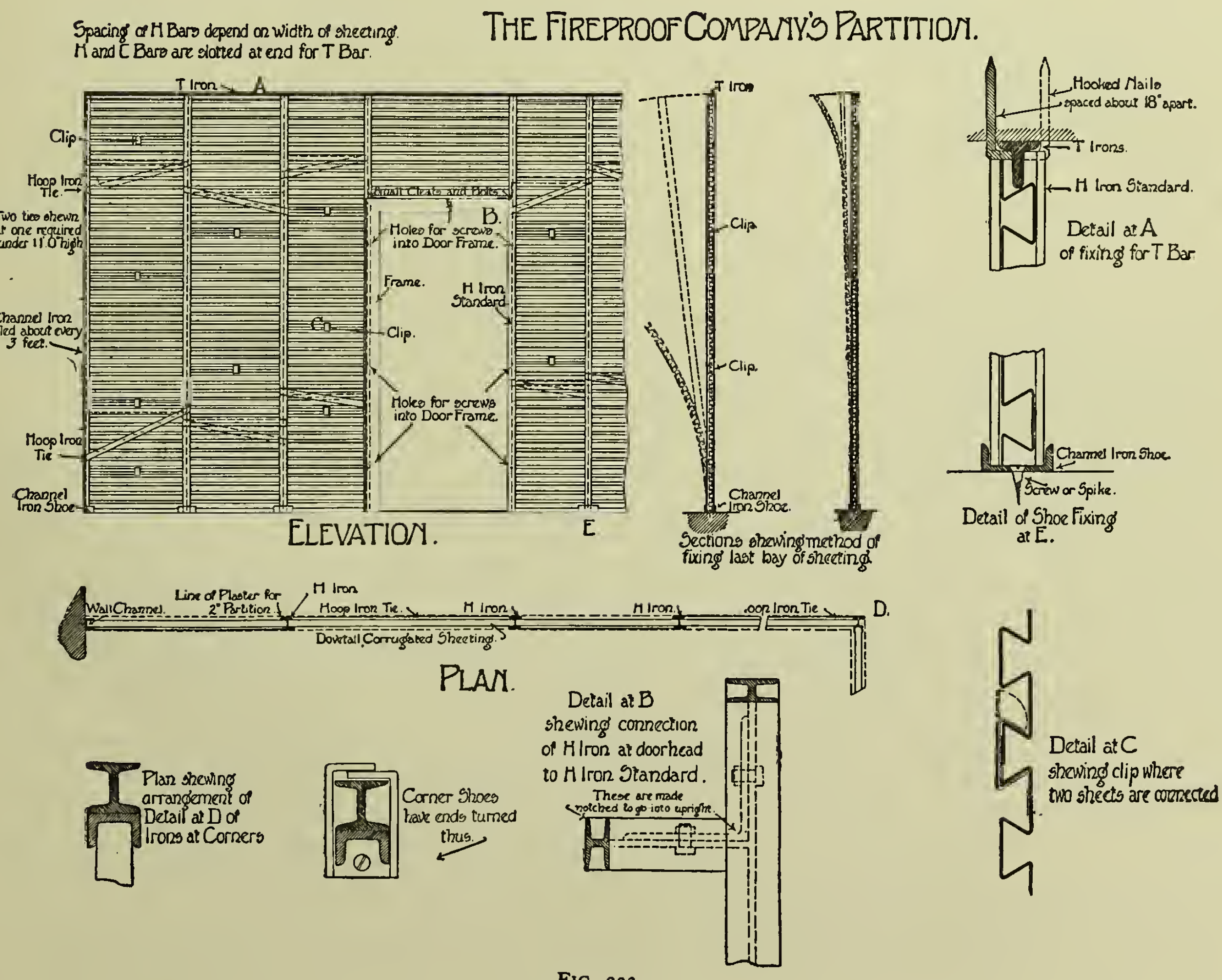


FIG. 233.

patent partitions have been produced, and though they will all obstruct the spread of fire, partitions which are less than 3 inches thick can scarcely be considered as thoroughly efficient fire-resisting divisions. Several patent partitions have been mentioned in another volume, but if they be thin and without metal reinforcement they can only be satisfactory for small areas,—say up to 40 times their thickness in height, and of length little in excess of this.

Jabez Thompson's Patent Terrawode Brickwood Partition is certainly of excellent fire-resisting quality.

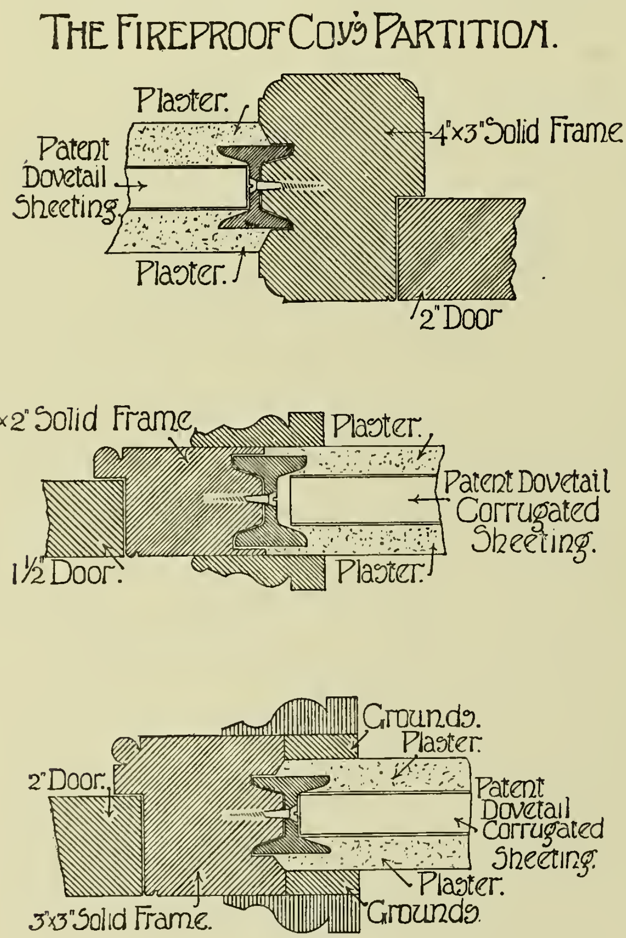


Fig. 234.

It is claimed by the maker that it is sound proof, and only half the weight of ordinary brickwork, while nails can be driven into it anywhere. It is built as is ordinary brickwork, with blocks of the same size as common bricks. Terrawode slabs for partitions are also constructed by this maker.

The Mack Partition, 2 to 4 inches thick, is also a good fire-resistant.

The Kulm and the Phœnix partitions are, again, satisfactory.

Hollow Terra-cotta. Tile partitions of thin structure will possibly confine a considerable fire, but the fire side of the tiles will break away from the webs, and the partition, as a rule, will become a total loss ; and this defect will occur with hollow blocks of any kind if of too thin a structure.

Partitions constructed of bricks or slabs should be wedged, with pieces of slate, etc., tightly against the under side of the floor above, in order to make them thoroughly rigid and firm.

The "Havelock" Patent Plaster Partition (Fig. 231).—This illustrates a type of thin partition formed with hollow stabs, which may or may not be strengthened with iron rods.

A Partition by the Expanded Metal Company (Fig. 232).—This is simply a solid plaster partition, the plaster being applied on either side of expanded metal, which is in turn supported by vertical rods. The total thickness varies from 1½ to 2 inches. Such a partition can be regarded only as a temporary stop to fire.

The Fireproof Company's Partition (Figs. 233, 234, and 235).—This partition consists of dovetail corrugated sheeting, held in H-uprights and plastered on either side,

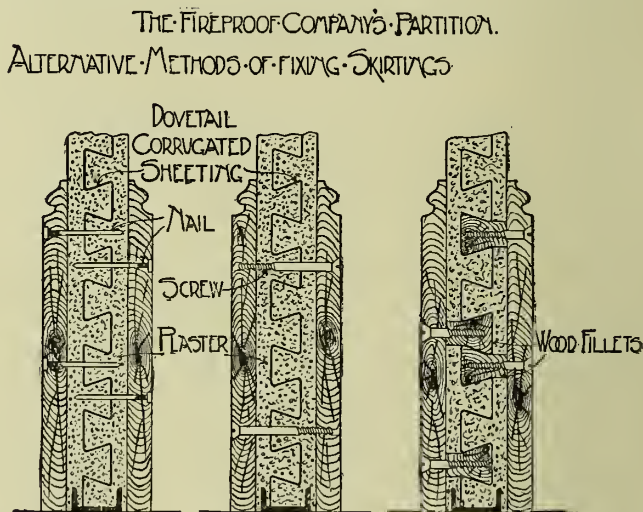


Fig. 235.

the total thickness being from 2 to 3 inches. It forms an exceedingly rigid partition. In case of fire the partition will become very hot, although no flame may pass. The plaster on the side towards the fire will be liable to fall or to be washed away by water. Figs. 234 and 235 show the connections of woodwork to the partition.

It is highly important that no wooden fixing strips shall be built into a partition, as these, on burning out, may totally wreck it. In America it has been a common practice to lay floor boarding over the whole floor area, and to erect the "fire-proof" partition upon this in the position desired by the tenant. The burning away of the flooring will consequently cause the collapse of the partition.

The use of any of the above considered partitions is not confined to use in buildings constructed on thorough fire-resisting principles ; but they are highly suitable in conjunction with wooden floors.

There are many more partitions on the market which will not be mentioned here.

Doors. — The efficiency of a good fire-resisting partition is of necessity greatly diminished by the provision of door openings in it. In fact, the flimsy door and linings so often provided may well act as a train for the spread of fire. But even the ordinary door, if shut and fitting well in its opening, may confine a considerable fire for the extent of a quarter of an hour or even longer. Yet, although such retardation is not to be despised, it is obviously out of place to provide such a door in a partition which is capable of altogether preventing the spread of fire.

Solidly constructed hard-wood doors are preferable, and a 2-inch well-fitting door of this description may confine a fierce fire for about half an hour. But greater protection than this is often necessary.

Iron doors have had a fair trial, and are now considered unreliable. Although incombustible, they warp and twist, letting flames through all round their edges. If they are used they require three hinges on the one side and three fastenings on the other, while even then they are not to be depended upon, and unless securely attached the fastenings may be pulled away by the warping of the door. Iron doors may, however, form an effective fire stop if used in pairs, one on either side of a small compartment in which no combustible material exists.

Tin covered doors, which were introduced to take the place of iron doors, are unquestionably superior, and, in fact, for sliding doors, these are at present the only fire-resisting doors recognised by the Fire Offices Committee. It is claimed of them that they never warp under the most severe heat, and never allow fire to be communicated by themselves becoming red hot.

A sliding and a hinged door by the Curfew Armoured Door Company are shown in Figs. 236 and 237, while a similar door by Messrs. Mather & Platt is shown in Fig. 238. These doors are constructed in accordance with the Fire Offices' Committee's specification, and are constructed of three or four thicknesses (according as to whether they are under or over 35 feet in area) of well seasoned deal boards, planed, tongued, and grooved, and nailed together, the nails being clenched on the farther side. They are then completely covered with tinned steel sheets with welted joints, the screws which attach these to the woodwork being beneath the welt and invisible from the outside. In some doors asbestos or uralite is used between the tin and wood. It will be noticed from the illustration that the hinged doors are made with two latches, which can be actuated at once by a single handle. The sliding doors are held to the wall on one side by a projecting angle, and by an iron roller on the other, which roller keeps the door close to the wall by engaging with a wedge fixed to the corner of the door. Details of the fittings of the Curfew doors are shown in Fig. 236. The doors should overlap their openings by not less than 3 inches on all sides, while a sill against the bottom edge is strongly to be recommended, which, besides preventing the passage of flame, will also stop water thrown by fire-engines from passing out and injuring stock. These doors, particularly the hinged doors, will generally resist a fierce fire for at least an hour; with the sliding doors there is a

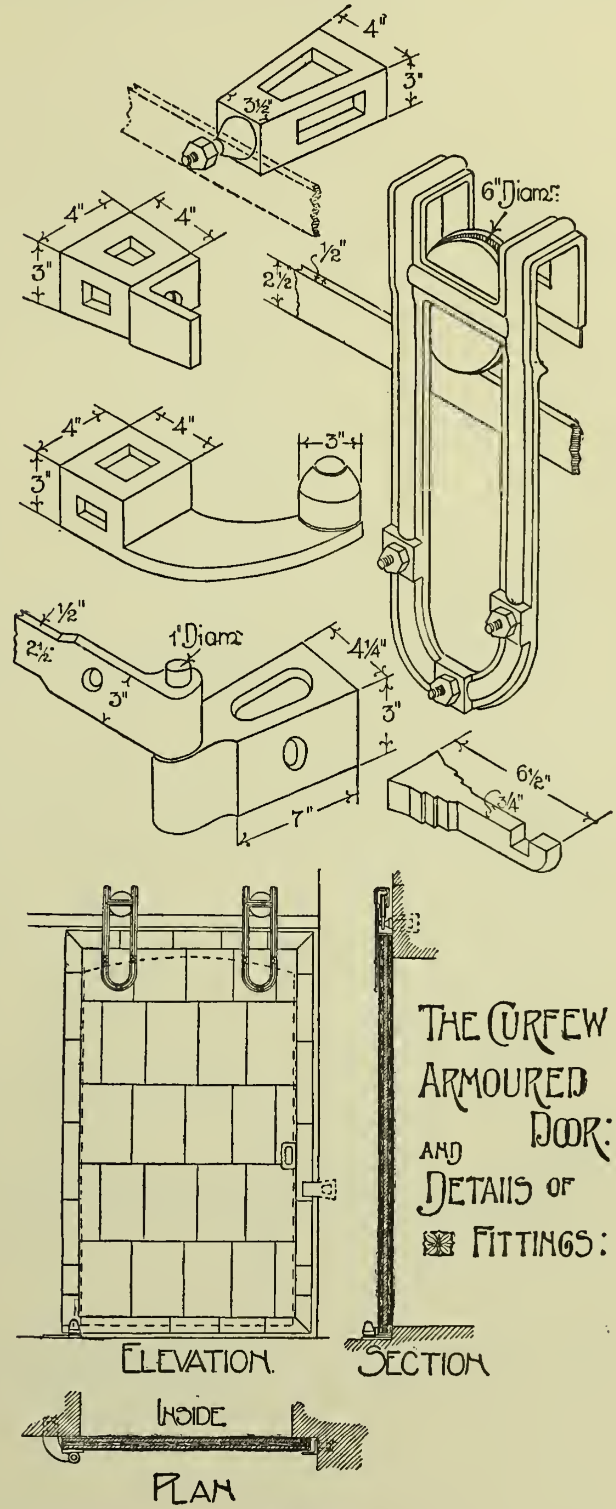


Fig. 236.

tendency not to fit tightly enough at the top edge, and consequently to allow fire to pass at this point.

Doors of the above description are obviously not sufficiently ornamental to be used in a number of cases

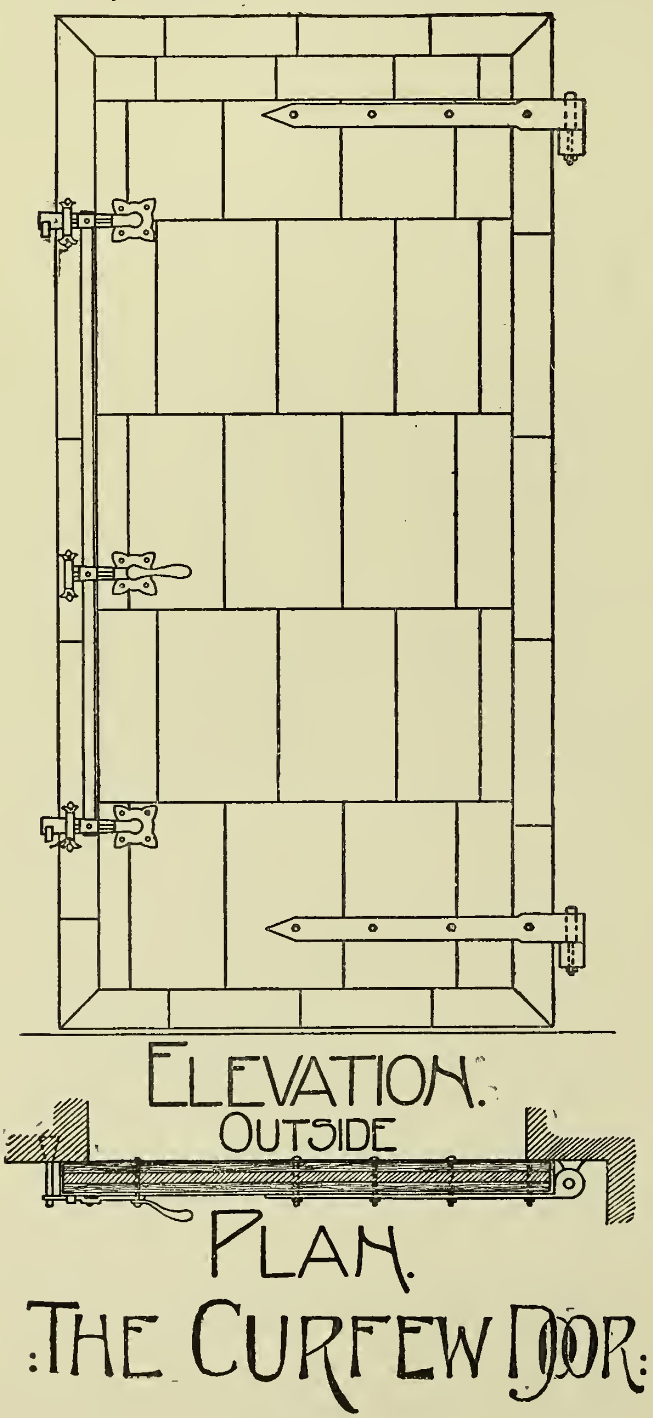


Fig. 237.

in which a satisfactory fire-resisting door is desirable. In America, it is true, metal-covered ornamental joinery of all description has been used, but it is found that doors constructed entirely of wood may be made almost as fire-resisting as those covered with metal, and at the same time are obviously more desirable for ordinary use.

Multiple Thickness Wooden Doors.—Doors made in three thicknesses, fastened together with wooden pegs, are found to resist the spread of fire remarkably well. Such a door, made $2\frac{1}{2}$ inches thick, should resist a fierce fire for three-quarters of an hour. Metal nails or screws should not be used in the construction of the door, as upon getting hot they burn the wood round them, and aid in its destruction.

Fig. 239 shows the construction of the Gilmore doors. The core of the framework of the panelled door, and the core of the whole of the solid door, is made up of strips of Canadian pine, $\frac{7}{8}$ inch thick, laid side by side and grain against grain, and glued together, the strips being placed vertically in the style, and horizontally in the centre of the solid door. Completely surrounding this core, asbestos board, $\frac{1}{8}$ inch thick, is fixed with 1-inch nails, while upon this a veneer of hard wood, $\frac{3}{16}$ inch thick, is glued and pressed by hydraulic pressure. At the time of glueing up numerous holes are made from top to bottom of the door, and wires are continually run through these with a view (at the time of the door being attacked by fire) to giving the gases in the wood egress, thus rendering the door more fire-resisting than would otherwise be the case. A panelled door $1\frac{3}{4}$ inch thick, with five panels $\frac{1}{2}$ inch thick, when tested by the British Fire Prevention Committee, withstood a temperature rising to 1500° or 1600° Fahr. for full 50 minutes without the passage of flame, while a solid door 2 inches thick withstood a similar temperature for 60 minutes without the passage of flame, thus obtaining classification under the head of "Temporary Protection," Class B (see Chapter I.). The door openings in both cases were 2 ft. 6 ins. × 6 ft. 10 ins. The frames were of 4 × 3 ins. oak, with $\frac{1}{2}$-inch rebates.

It is of primary importance that a door should fit well in its frame, in order that draught may not pass between, for if draught passes fire will also pass, and the door will be attacked at probably its most vulnerable points, namely, its edges and corners. To avoid this, the use of deep rebates in the door frame is to be strongly recommended, as is also the employment of a sill against which the door may rest.

In the case of tin-clad doors no frame is used, but the door is given a lap of not less than 3 inches over the sides of the opening.

The fastenings, hinges, and latches in wood doors are liable to be a source of weakness, for on heating they tend to ignite the wood and to become loose. They must, however, be fixed as firmly as possible; latches should be bolted right through the door, while hinges should be bolted and not screwed, to both the door and the door frame.

The provision of fire-resisting doors will be of little use if, on the occurrence of fire, these be all left open; but the closing of them must be largely left to the intelligence of the inhabitants. The closing of a door

not only hinders the spread of fire, but also, by excluding the air, diminishes its fury. In important cases the closing of a door may be done automatically by the burning of a cord and the release of a counterweight. This arrangement may be seen in the illustration of a sliding tin-covered door (Fig. 238); while hinged doors may also be made to close automatically. In the case of warehouses, manufactories, etc., all doors should be closed after working hours.

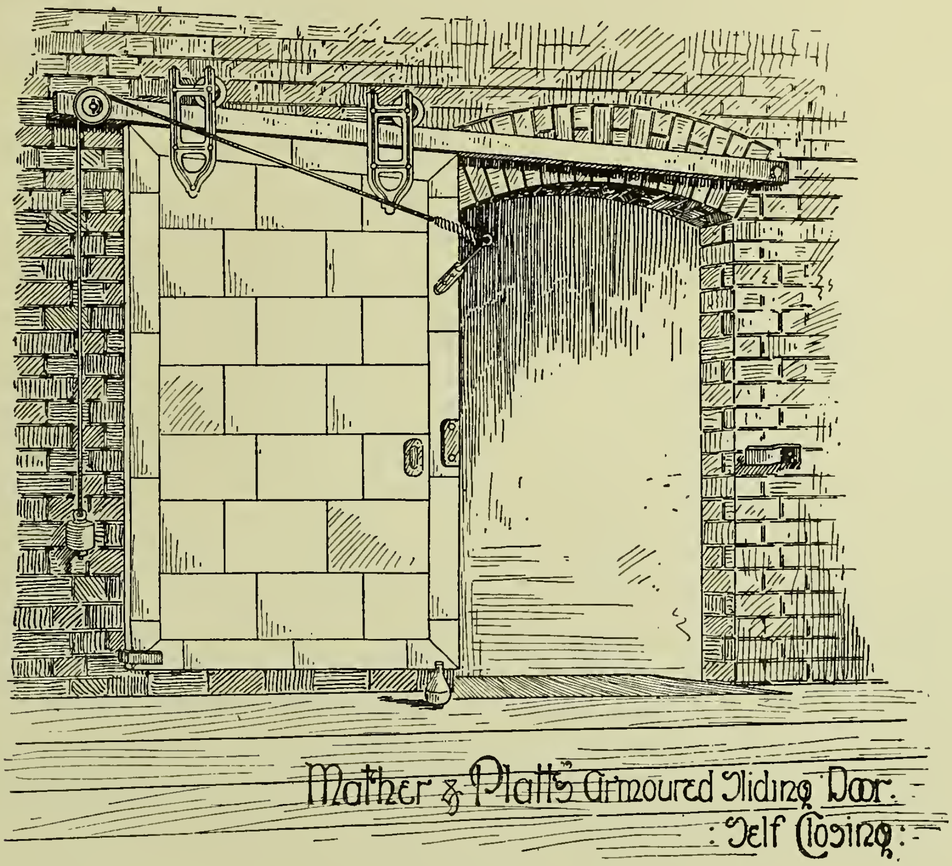


FIG. 238.

For tests upon many forms of door, as well as upon floor partitions, and other details, the reader is referred to the reports of the British Fire Prevention Committee.

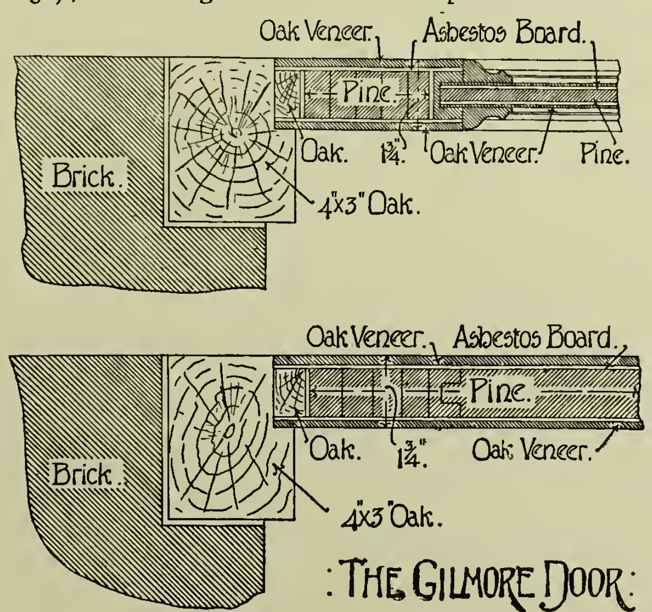


FIG. 239.

CHAPTER IV

FIRE-RESISTING CONSTRUCTION: STAIRS AND EXTERNAL RISKS—WAREHOUSES

Staircases.—It has been stated that, having provided fire-resisting floors, no opening should be made in them; but it is unfortunately impossible to avoid this entirely, for staircases and lifts must be provided, and in the majority of cases these are disposed in the very heart of the building. In these shafts the volume of air is large and constantly moving, a condition which much favours the creation of a fierce fire, and, unless careful provision is made, it will be the means of spreading the fire to every floor in the building.

If an opening to the outer air be made in the top of this stair shaft the heat of a fire, together with an open door at the foot, will create a strong draught as in a

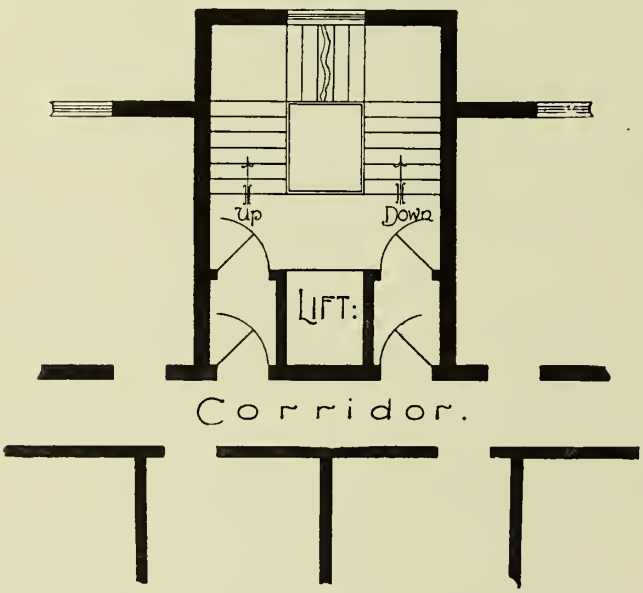


Fig. 240.

chimney, and the fire may by this means be spread to every floor almost simultaneously. It is therefore held by some that the head of such a shaft should be covered with some fire-resisting material which will prevent this up-draught; but even if this be done it will not prevent the spread of fire upwards, while the products of combustion will immediately collect in the upper part of the shaft and completely negative the efforts of people either to effect their escape or to attack the fire in this direction; for it may be mentioned here that probably the majority of fatalities from fire are primarily caused by persons being overcome by these fumes and not by the actual heat of the fire.

Not only will the covering over of this shaft fail to prevent the spread of fire, but it may cause a draught to be produced in another direction, and so actually aid the fire in attacking other parts of the building. If, however, the top of the shaft be open to the air, the fumes will pass away, while if each floor be screened by fire-resisting doors at every landing the fire also may pass almost harmlessly up this flue. In order that the shaft may be open at the top in case of fire, the skylight over it may be glazed with thin glass, which will crack directly the fire reaches it, while it should also be possible to readily open it by hand to allow smoke to escape. It is recommended by some that in order to prevent the creation of a draught the skylight should be glazed with fire-resisting glass; but that it should be capable of being easily opened by hand. However, thin glass will not crack and create a draught until the fire reaches it, and it would then be impossible to open the skylight unless the mechanism employed extends down to the lower floors of the building.

The walls surrounding the shaft must be carried up of fire-resisting material through the roof, and projecting above it.

The cutting off of the stair shaft with doors at all floors may not always be convenient, but it should be one of the first objects in designing a building which is intended to be thoroughly fire-resisting. Fig. 240 shows a plan in which every floor is cut off from the stair and lift shaft by double doors separated by a fire-resisting compartment. By this arrangement fire on one floor is completely cut off from the shaft, while it is open to persons on other floors to make their escape or for firemen to enter; while if a door be provided at the bottom of the shaft, access may be obtained directly to the open air without having to pass through the ground floor.

Where possible there should be no well hole, the flights of steps being placed on either side of a central newel wall built up from the ground, as in Fig. 241. This arrangement obstructs the direct passage of fire; while the steps, being well supported at either end, are rendered less liable to failure from the effects of fire.

There should be, if possible, nothing within the staircase shaft that is capable of adding to the flames, and woodwork, other than hard wood, such as oak, teak or jarrah, solidly built and at least 2 inches thick in all places, should be strictly avoided. The framing of a staircase must be of iron, while if an open staircase is

to be employed the steps must not overhang, but must be supported right up from the ground. The treads of the stairs may preferably be of concrete. Solid stone should not be used, as this will be unreliable and dangerous to firemen. If stone slabs are used for the treads they should be supported by cast-iron grids, so that, in the event of the stone disintegrating, the stairs will still be passable.

The internal light shaft that is often to be found in the larger shops and other places forms a particularly ready means for the spread of fire from floor to floor. These shafts should be strictly avoided, but where they exist the openings in the floors should be provided with sliding or rolling shutters, which may be drawn horizontally across them in case of fire, while they should always be closed at the conclusion of working hours.

From what has been said it is evident that a fire, on

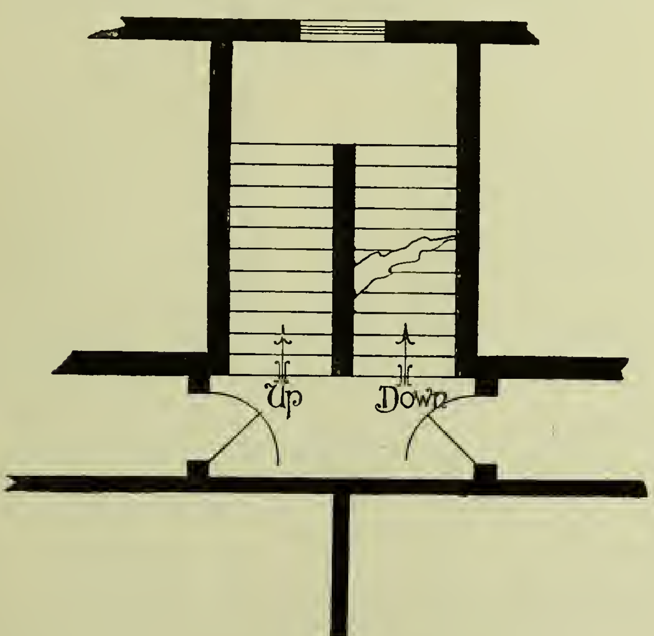


Fig. 241.

assuming large proportions in any floor of a building, will almost certainly attack the stair shaft, and means of escape in this direction will be cut off. It is therefore essential that some further means of escape shall be provided, and this may perhaps best be done by the use of another internal staircase as far away from the first as possible. The two staircases should preferably be placed at opposite ends of a building, which should be so planned that either staircases can be reached from all parts.

A second means of escape may be obtained by situating a lift at some distance from the staircase, although, if the motive power be affected by the fire, it is rendered useless. Where a lift is separated from a staircase, which should always be arranged where possible, it should be enclosed in a brick shaft, the opening at each floor being filled with a fire-resisting door, which door may be furnished, if desired, with a window of wired glass. The opening to a freight lift may be automatically cut off by a fire-resisting door, such as that shown in Fig. 242.

The internal staircase may be supplemented by an external iron staircase, but this has the objection that it may be put to improper use by burglars and others. Such a means of escape should take the form of a staircase of easy pitch furnished with hand-rails, for it is to be remembered that it is intended for the use of frightened and possibly panic-stricken people. A staircase of this description must not lead down to a closed courtyard, for in such a position people may be trapped

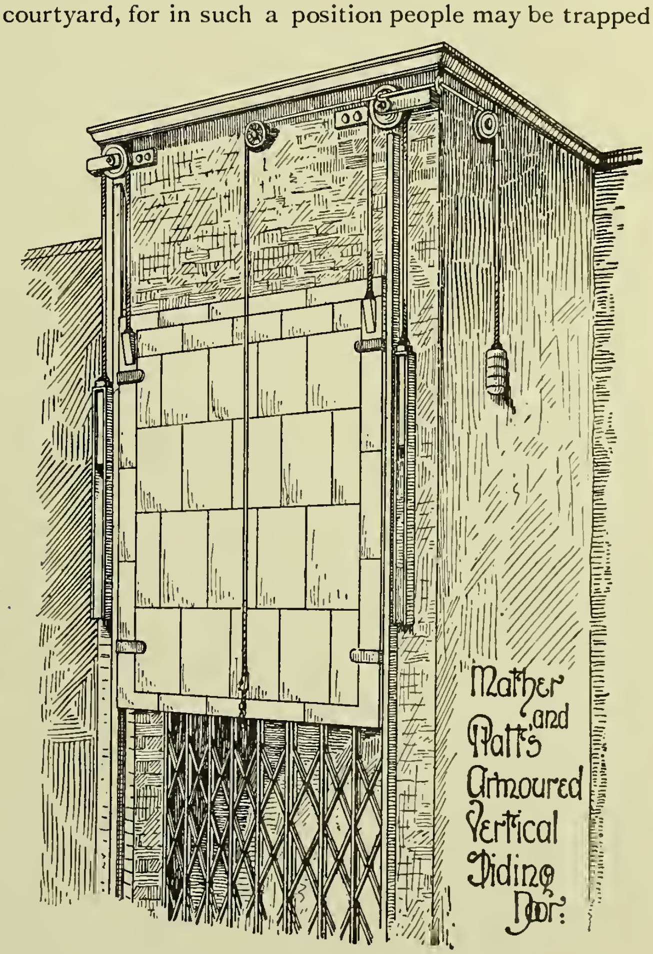


Fig. 242.

and overcome, but easy communication must exist with the street

The provision of means of escape to roofs is strongly advisable, and the London Building Act Amendment Act 1905 specifies that all buildings being more than two storeys above the ground storey, or exceeding 30 feet in height, shall be furnished with some form of ready access to the roof; and a sufficient parapet or guard-rail to prevent people from slipping off the roof is to be provided where necessary.

The upper floors of buildings of greater height than 60 feet are out of reach of the London Fire Brigade's ladders, and in this case special care must be taken to allow of adequate means of escape.

External Risk.—Up till now only the internal division of a building and the confining of internal risk has been dealt with ; but there is still another serious risk which may practically nullify the most thorough internal division of a building. This danger is the "external risk," or the risk of fire being communicated from an external conflagration through the window openings, and also by means of the roof if this is not of fire-resisting construction. The necessity for the thorough fire-resistance of the external and party walls has already been considered, but the value of external walls for this purpose may be completely destroyed by the provision of large and unprotected window openings. Through these openings fire from an adjacent conflagration may enter every storey of a building almost simultaneously; while, as has not infrequently been the case, a great conflagration, although barred from direct passage from one house to another by means of unbroken party walls, may continue its path by crossing and recrossing streets and entering the buildings through the windows. As heated air and flame must rise, it is the upper storeys of a building which are most liable to suffer from external causes, and the risk of fire crossing streets is much lessened by regulations governing the height of buildings in proportion to the width of the streets. However, in the older parts of our cities and towns, buildings are arranged with quite inadequate spacing between them, behind as well as in front.

Small areas completely enclosed by buildings, as illustrated in plan (Fig. 243), are particularly dangerous. The space between the buildings here shown is totally inadequate, and the spread of fire, when it has once got a hold, is almost certain, although each building is divided from the one next it by fire-resisting walls, as indicated in thick lines. To make the matter worse, the area is often roofed over with glass, covering rooms which have free access to the interior of the surrounding buildings ; and the falling of burning timber through this glass roof will quickly cause the spread of fire, while at the same time the enclosed area may be rendered quite inaccessible to firemen. A fire, when it has once taken hold of such a block of buildings, and given a favourable breeze, will find little difficulty in spreading across the streets to other buildings, and its arrest may be exceedingly difficult. Many such areas exist in London and in other cities, and a process of thinning out is as necessary from the point of view of fire-resistance as it is from that of sanitation.

Light wells, external or internal, form a ready means for the spread of fire, and the window openings round such shafts are in special need of protection.

Not only is protection to window openings necessary to guard against risk from adjacent buildings, but fire which is prevented from progressing upwards by fire-resisting floors may make its way throughout the height of a building by coming out from one window and entering at a window in a floor above. Again, the existence of an unbroken protection will keep out air and diminish the intensity of the fire, besides lessening the external risk to other buildings.

It is thus seen that protection to window openings is very necessary, and particularly so in light wells and other dangerous places.

The first safeguard in this direction is that no window should be made any larger than is necessary. The opening may then be protected either by fire-resisting shutters or by fire-resisting glass, or still better by the conjunction of the two.

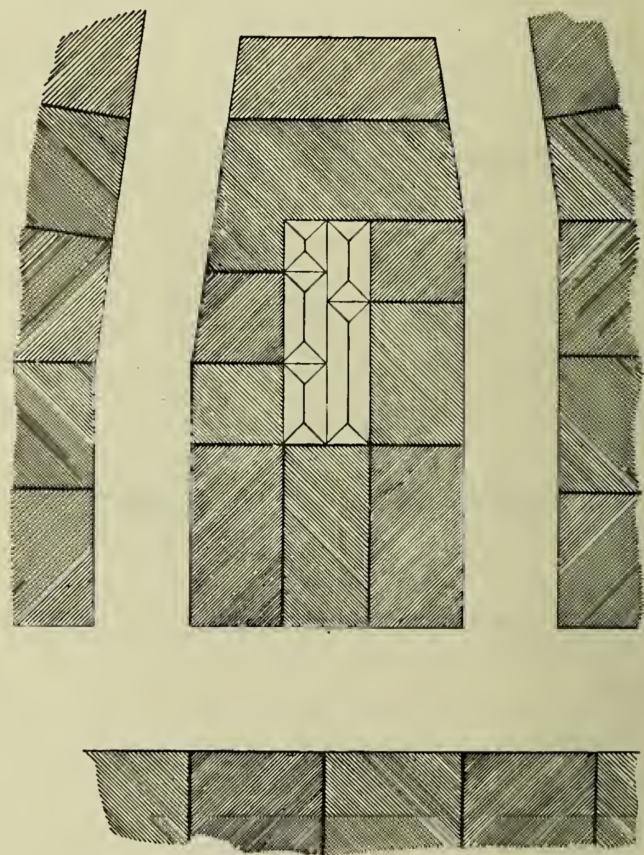


Fig. 243.

Shutters, made precisely similar to the tin-covered doors previously mentioned, may well be used, especially where the risk is great. Plate XII. shows a number of such shutters erected by Messrs. Mather & Platt. The shutters against the farther wall are opened and closed by simultaneous gear, as can be seen in the photograph. Where such shutters are provided and allowance is made for their use, a guarantee that they shall be closed after working hours is required by the insurance companies.

In some positions, on account of their appearance, shutters are deemed objectionable, in which case iron roller shutters may be substituted.

It is, however, evident that the use of shutters of any description is hardly feasible in domestic and office

FIRE-RESISTING SHUTTERS, PROTECTION FROM EXTERNAL RISK (MATHER & PLATT).

buildings, for even if they be provided a warrant can hardly be given that the separate tenants of offices will close their shutters before leaving. It is, though, fortunately possible to afford temporary protection to window openings without the aid of shutters by means of wired glass or electro-copper glazing.

Wired Glass, by Messrs. Pilkington Bros., is illustrated in Fig. 244. It is made $\frac{1}{4}$ inch thick, and, as seen in the illustration, contains wire netting of $\frac{7}{8}$-inch mesh embedded in the centre of its thickness.

Two horizontal openings 2 feet × 2 feet and two 2 feet $1\frac{1}{2}$ inch × 2 feet $1\frac{1}{2}$ inch, as well as three vertical openings 2 feet 3 inches × 4 feet 6 inches, two of the latter consisting of two squares and all glazed with glass of the above description, were tested by the British Fire Prevention Committee and fully qualified as "Temporary Protection, Class A."

This glass may be obtained in three forms—"Patent Cast Wired Glass," "Patent Rolled Wired Glass," and "Patent Polished Wired Glass," all $\frac{1}{4}$ inch thick.

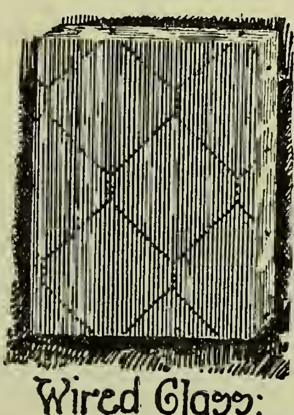


Fig. 244.

The last of these three is transparent, and could without hesitation be used for office windows.

Electro Glazing by the British Luxfer Prism Syndicate is illustrated in Fig. 245, and, as there seen, may be plain or of any ornamental pattern. When required for fire-resisting purposes it is generally formed of squares or other shapes of plate glass $\frac{1}{4}$ inch thick, mounted or united by electro-chemically deposited copper, thus producing intimate contact between the copper and the glass. The glass connected in this way may be of any description, while Luxfer prisms are similarly treated. Three windows of two lights each, each light measuring 2 feet × 2 feet, tested by the British Fire Prevention Committee, obtained classification under the head of "Temporary Protection, Class A."

Ordinary glass, when subjected to fire, will crack and fall almost immediately. The two forms of glazing just considered will similarly crack when subjected to fire, but will not give way even when water is discharged against them from a fire hose. Wired glass, at any rate, is said to withstand the passage of flame until the glass melts. It is advisable, when using fire-resisting glass, to limit the size of the squares to about 2 feet × 2 feet. It should be remarked that, although the above-mentioned glass will resist the passage of flame, yet a considerable amount of heat may be transmitted through it.

Window frames to hold fire-resisting glass must of course be equally fire resisting. Hard-wood solid frames may be used if of moderately substantial proportions with no slender bars, while steel or iron frames are perhaps rather more satisfactory.

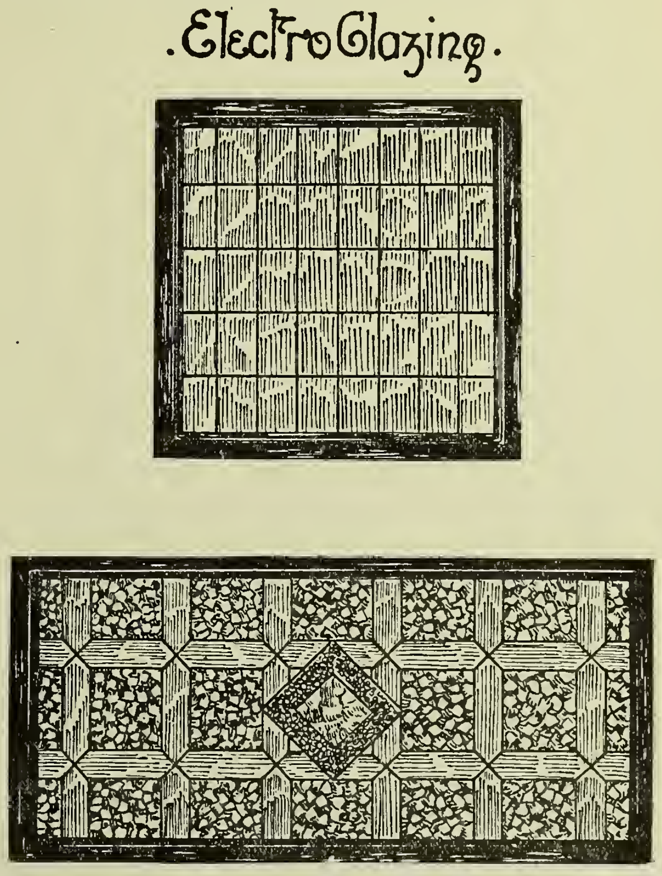


Fig. 245.

Ordinary putty must not be employed in fixing the glass, for it is liable to burn. The two forms of glass above mentioned are fixed with beads and a cementing material of asbestos.

Internal Woodwork.—Having provided a fire-resisting shell, it is expedient that as little material as possible of an inflammable nature shall be introduced. Woodwork is practically a necessity in most kinds of buildings for floors, skirtings, architraves, etc.; but with care the danger of its rapidly producing a fierce fire may be greatly reduced.

Wood may be chemically treated to render it non-inflammable. This treatment does not prevent the wood from being consumed, although this takes place con-

siderably more slowly than is the case with untreated wood. The great value of treated wood lies in its refusal to support combustion. Treated wood will glow and char when placed in a flame, but will not itself produce flame. Joinery thus treated may now be obtained at only slightly increased prices. Treated wood is, however, liable to certain failings. There is a tendency for the wood to be hydroscopic, and also to have the effect of causing the corrosion of metals. Certain processes are claimed to avoid these defects, while the objection can be surmounted by painting the wood. A coating of paint or varnish appears in no way to increase the inflammability of treated wood, which may be considered as another point in its favour. A further effect of the treatment is a liability to render the wood slightly brittle, but for many purposes this is of no importance. The effect of the process appears to be permanent, but this cannot as yet be definitely asserted.

Beyond the use of non-inflammable wood much may be done by careful construction to render wood less ready of ignition and slower in burning. The use of thin wood with projecting edges is particularly to be avoided, while such architraves, etc. as are necessary should be of a solid description, and preferably of hard wood.

The chief danger in the use of wooden finish lies, however, in the practice of fixing wainscoting, skirting, etc. to projecting grounds, with an air space between wood and wall, producing very favourable conditions for rapid combustion; while, if the spaces between grounds or fixing strips were completely filled with plaster, even thin wood might be used with greatly reduced risk. In buildings, however, in which it is intended that the fire risk shall be reduced to the minimum, the use of wainscoting should be entirely out of the question, while the skirting may be formed in cement.

Above all things, air spaces behind woodwork are to be carefully avoided, and this applies hardly less to the case of flooring than to other wooden finish. It is generally desirable to be able to carry piping, etc. underneath the flooring, and to allow of this it is usual to fix boarding on the top of wooden strips, leaving a space of from 2 to 4 inches between the concrete and the boarding. This space must be filled in, and a poor mixture of cinder concrete may well be used for the purpose. A poor mixture is advisable in order that it may be readily removed when necessary; and if this be done at any time, in order to allow the insertion of pipes, etc., it is important to see that this filling is afterwards returned to its place. In many systems of flooring the top flange of the joists is left projecting above the concrete, while the wooden flooring is raised up above this again; an arrangement which, in case of fire, will probably result in the overheating of the steel and the distortion of the floor.

Warehouse and Mercantile Buildings. — The principles hitherto discussed apply to all forms of buildings. Buildings of the warehouse class, however, are subject to special risks which must be considered separately.

The intensity of a fire is governed to a great extent by the cubical contents of the area in which it burns. Thus in very large areas the intensity of a fire is likely to be considerably greater than in a small area. The London Building Act limits the undivided contents of a warehouse to 250,000 cubic feet; but a warehouse may be made of any extent, provided it is divided up by thoroughly fire-resisting walls. The rules of the Fire Offices' Committee limit the contents of any one compartment in all buildings, except cotton and similar mills, to 60,000 cubic feet; while in cotton mills, flax mills, etc. the superficial area of any one compartment may not exceed 25,000 square ft. By dividing a large area with fire-resisting walls, the risk of loss by fire is correspondingly reduced, while the intensity of a fire in any one compartment is also lowered. Such dividing walls should be carried up at least 3 ft. above the roof.

It will generally be necessary to have communication between compartments, and openings may be made in the dividing walls, provided that they are thoroughly protected with double fire-resisting doors. The rules of the Fire Offices' Committee allow communication between compartments, the total area of which taken together exceeds the maximum dimensions given above, only by double fire-resisting doors on either side of a fire-resisting compartment, the doors being at least 6 ft. apart. These doors should always be closed when not required for use. Automatic attachments to close the doors in case of fire would be highly advantageous.

The danger of staircases, as previously mentioned, in communicating fire from floor to floor, is of great importance in a warehouse, and the risk should be guarded against as far as is practically possible. The staircase should be in a fire-resisting shaft with central brick newel wall, only communicating with the compartments through fire-resisting doors. Another way of lessening the danger is by the exclusive use of external staircases, with, perhaps, the addition of balconies to communicate with the various compartments of the building.

In manufacturing buildings, where many hands are employed, special provisions for escape must be made. (See Remarks with regard to escape, in chapter on "The Protection of Theatres" in Vol. VI.)

The use of scuppers in external walls is advisable in order to allow water thrown by fire-engines to run off, instead of flowing to other parts of the building to the injury of stock in floors below.

CHAPTER V

PROVISIONS FOR THE EXTINCTION OF FIRE

In order that a fire may be extinguished at once, its immediate detection is essential, and in all cases of special risk, such as theatres, warehouses, etc., men should be specially employed to "watch," or to make complete periodic tours of inspection over the whole building. In warehouses and factories, where watchmen are employed at night whose duty it is to make tours of inspection at stated intervals, some mechanical device should be used to check the times at which the various parts have been visited.

Means of extinguishing fire in its incipiency, such as fire buckets, chemical extinguishers, hand pumps and hydrants, should be provided in as many positions as is necessary, and should be so situated that they may be immediately applied at any point in the building. A further protective measure, particularly on the stage of a theatre, is the provision of rugs or blankets, preferably wet, with which an incipient fire may be smothered. Should such appliances as are above mentioned fail to have immediate effect, outside aid should be sent for at once before exerting further efforts to overcome the fire, while it should be observed that all doors, windows, or other openings are closed.

Automatic Alarms.—In certain cases a thorough system of watching is impossible, while even where a night watchman is employed the thoroughness of his tour of inspection cannot be ensured. These difficulties may be surmounted by the employment of a trustworthy system of automatic watching,—that is to say, by the use of apparatus which, on being raised to a certain temperature, or on a sudden rise of temperature, will, by completing an electric circuit, sound an alarm in a central office, indicating on which floor the outbreak has occurred, as well as its exact position on that floor. These apparatuses are set in action by the expansion of metals or gases, by the fusing of a solder, or by the varying expansion of two metal strips, which thus make an electrical contact on being heated. They should be placed on the ceiling about 15 feet apart, or one such apparatus to every 225 square feet. In their position upon the ceiling they will quickly be affected by any appreciable rise in temperature. The employment of these automatic alarms is particularly desirable in hotel buildings.

Sprinklers.—An automatic arrangement which will give the alarm of fire besides automatically aiming at its immediate extinction is particularly desirable, especially in the case of warehouses and factories containing inflammable goods. This desideratum is met by the automatic sprinkler, in which a rise of temperature will open an automatic valve, and water will be showered on the floor below in the manner represented in Fig. 246. These automatic nozzles, or "Sprinkler Heads," are placed 8 to 10 feet apart, and thus command 64 to 100 square feet of floor space. A complete installation of sprinklers requires an extensive system of piping.

In certain cases a rebate amounting to as much as 50 per cent. may be allowed on insurance policies where an installation has been applied. (See the rules of the Fire Offices' Committee concerning Sprinklers.)

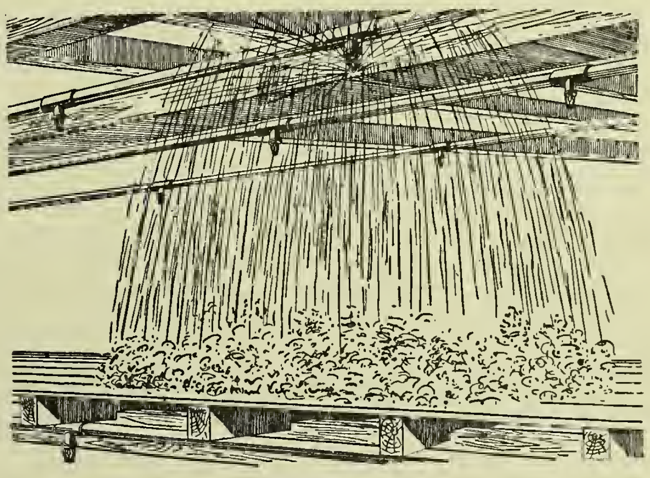


Fig. 246.

Sprinkler Heads. — Fig. 247 shows the Grinnell sprinkler head of Messrs. Mather & Platt. The $\frac{1}{2}$-inch diameter opening in the head is closed by a hemispherical disc of glass, kept in position by a strut formed in three parts soldered together. On being raised to a temperature of 155° Fahr. the solder melts and the strut collapses, resulting in the opening of the valve. The water flowing in a $\frac{1}{2}$-inch stream impinges against the plate below it, and is in turn sprinkled up against the ceiling and down on the surrounding floor (Fig. 246). It may be noticed that although the solder will melt when raised to 155° Fahr., yet the surrounding temperature will be rather higher than this before the valve is actuated, perhaps 180° to 200° Fahr.

The requirements of a good form of sprinkler head are that it shall be extremely sensitive, and that its

sensitiveness shall not decrease with age ; also, that the opening of the valve when once started shall be complete.

The advantages claimed for the particular form of sprinkler illustrated are as follows. The valve being of glass is non-corrodible, non-adhesive, and impenetrable ; while, seated on the edge of a german silver diaphragm, it is perfectly free to open when released. The diaphragm, moreover, being held in a state of tension, is made to exert a constant positive force, more effective than 200 lbs. of water pressure, which, when the solder melts, severs the valve from its seat and thus overcomes the acknowledged danger of failure from the adherence of the valve to its seat. The water pressure on the diaphragm also helps to force it against the valve. A further purpose of the flexible diaphragm is to cause the valve and its seat to move outwards simultaneously until the soldered joint is completely severed. Were not the orifice kept closed until the solder joint has broken entirely, a slight escape of water would cool and reset the solder when the valve commenced to open, and thus prevent the proper working of the sprinkler. The construction of the strut prevents its gradual yielding or the accidental rupture of the fusible solder.

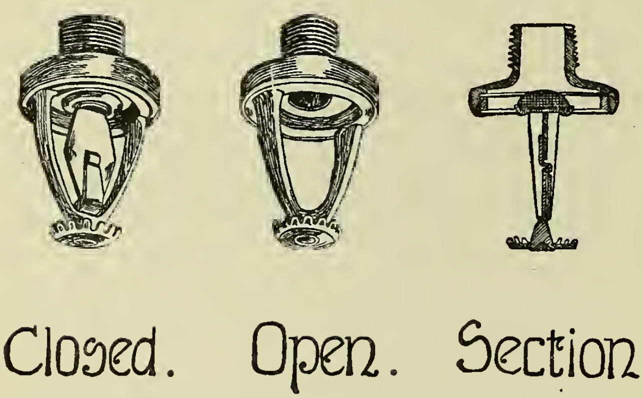


Fig. 247.

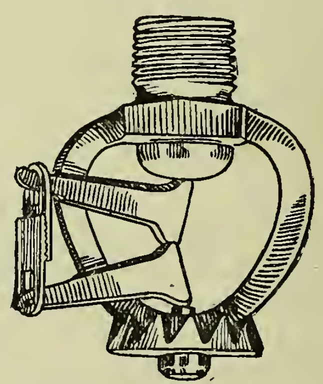


Fig. 248.

Fig. 248 shows the " Morris " sprinkler, which illustrates a type of sprinkler head in which the solder-bar stands out well away from the water drip.

A Sprinkler System, to be complete, must govern every portion of floor space, and care must be taken that out-of-the-way corners are not overlooked, for these are perhaps more likely than other parts to produce an outbreak of fire. Fig. 249 illustrates an installation of Grinnell sprinklers.

The opening of a sprinkler head should immediately sound an alarm at some central point, and this is actuated by the flow of water in the pipes. This alarm should continue to sound as long as the water is running. The necessity for this alarm is, in the first place, to call attention to the existence of fire ; and secondly, to ensure that the water may be shut off and prevented from doing damage when the sprinkler head has been opened either accidentally or from some slight cause.

Source of Supply.—In order that sprinklers may at all times have a source of supply, and may not be rendered useless by the temporary shutting off of the water from a main, a large tank should be provided, and this tank, as well as all pipes in connection with the sprinklers, should be reserved to their exclusive use. This may well have two alternative sources of supply, which may be opened to the sprinklers before the contents of the tank is exhausted, while means may also be provided by which the system may be connected to the hose of a fire engine. The sources of supply may be the public water supply, a private reservoir, or any practically inexhaustible supply. Where the reservoir or other source is not situated above the level of the tank, the water must be pumped up with a pump kept solely for fire-service. The tank need not be of very great size, nor need it vary quite in proportion to the number of heads installed ; for if the sprinklers are to extinguish a fire, they will probably do so within ten minutes, while if more than ten or fifteen heads are called into action, the fire probably will then have got beyond their control.

Dry-pipe System.—In situations where the water is liable to freeze in the pipes, a system known as the " Dry-pipe System " is employed. Water is prevented from entering the pipes by means of a valve, which valve is kept closed by filling the pipes with air under pressure. Upon a sprinkler head being automatically opened the air in the pipes escapes, and water is allowed to enter them and to shower out from the open head.

Open Sprinklers. — It is not necessary that the sprinklers should be automatic in action. The sprinkler heads may all be open, while the water is turned on by hand in case of fire. A system of this description is particularly suitable over the stage of a theatre, where its use is most necessary and should be universal. Every part of scene stores, shops, and dressing-rooms, etc. in connection with a theatre should be governed by automatic sprinklers, while a row of open sprinklers, so arranged that they will cause a sheet of water to flow down the surface of the fire screen, will tend to keep the latter cool and will greatly enhance its value.

Sprinklers used Externally.—As a protection against external risk, sprinklers are also valuable. Open

sprinklers may be used to cover a roof with a sheet of water, while others may be fixed along the cornice, producing a curtain of water down the whole face of the building. In high buildings this may be supplemented by another row half-way down the wall. Another system, or one which may be used in conjunction with the above, is to protect every window opening with a sprinkler at its head. Sprinklers in such positions are a most important safeguard against external risk, and should be classed with tin-covered shutters and fire-resisting glass. The system is preferably not automatic, but should be controlled by hand for use in the event of an external fire in dangerous proximity.

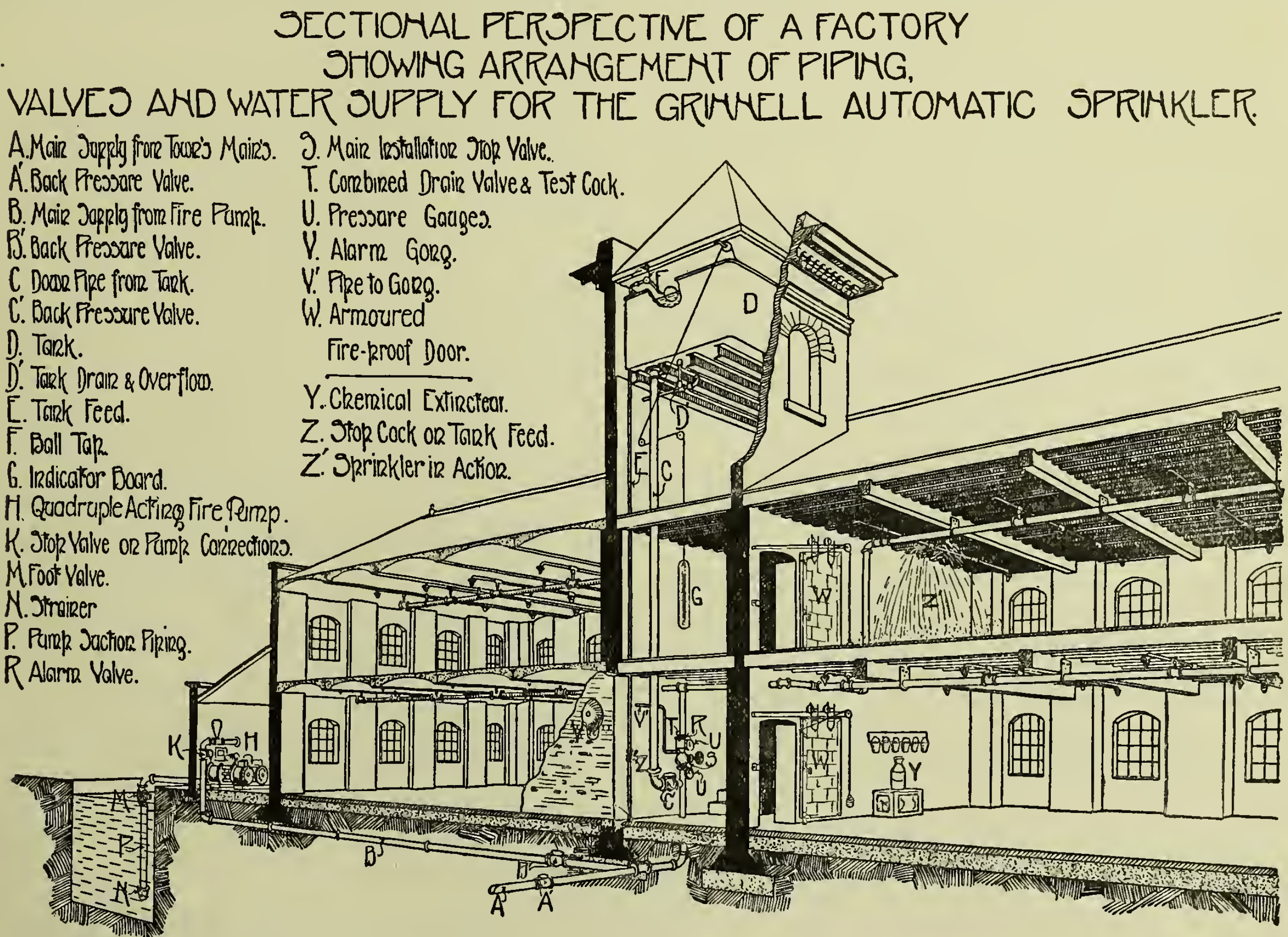


FIG. 249.

Where a large window opening, protected by a rolling shutter, exists in a position which is exposed to external risk, its condition may be much improved by fixing sprinklers behind the shutter, and between it and the glass, thus keeping them both in a cool condition. (*Note.*—The Fire Offices' Committee's rules state that no opening exceeding 50 superficial feet will be deemed capable of efficient protection by shutters.)

TALL BUILDINGS.—The American "sky-scraper" or any similar building presents further difficulties in fire extinction, for a fire-engine will not throw water much above 100 to 120 feet, and above this height a fire must often be left to burn itself out. Hosing may be taken up the staircase, but the length required and the time taken in the procedure are great objections, although the use of a lift will much hasten matters. In such cases a large rising main is necessary throughout the height of the building, with hydrants at each floor, and with a connection for fire-engines at the street level, and should be temporarily supplied with water from tanks at the top of the building. By these means water may be thrown upon a fire in any floor of the building, as long as fire and smoke do not entirely prevent access to it. Should the staircases and lifts become impassable, the people in the upper floors will be doomed unless there is still access to an external iron staircase. Apart from the question of light and air, tall buildings are seen to have serious objections in view of the possibility of fire.

In London the height of buildings is limited to 80 feet *plus* two storeys in the roof, while the London Buildings Act requires that any floor more than 60 feet from the street shall have some special means of escape; for 60 feet is the limit at which the London fire-escapes are of service. Escape on to the roofs of other buildings may often be arranged, or an external iron

staircase may be carried down to the ground, being in connection with each floor. The latter is a very desirable feature in any building in which many lives are concerned, while it has the further advantage of aiding the fire-brigade in their work.

SMALL HOUSES.—Before concluding the subject of fire-resisting construction, common house constructions may be touched upon. In the ordinary case of a small house with hollow lath and plaster partitions, floors of light timber construction, and thin joinery fixed to walls upon projecting grounds, the possibility of fire can never have been considered. Much may, however, be done to lessen the rapidity with which a fire may progress in such a structure without any very great additional outlay.

The use of lath and plaster partitions should be avoided as far as possible, but if their use is necessary for the sake of lightness they may be filled in with

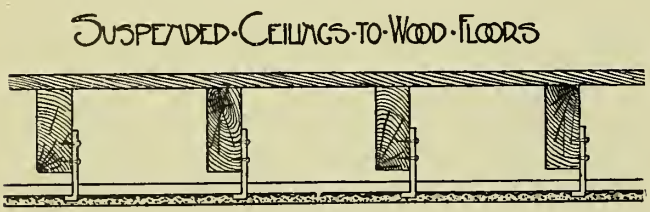


FIG. 250.

slag wool, while metal lathing may be substituted for wooden lathing. However, the use of some light moderately fire-resisting patent partition is to be preferred.

A suspended ceiling with metal lathing is illustrated in Fig. 250, and this alone will considerably retard the progress of fire. Floors also may be filled with slag wool, while such materials as silicate cotton, wire-net felting, or fibrous plaster and asbestos slabs may be attached to the under side of the joists, or fixed in between them with advantage. The use of grooved and tongued floor boarding, by preventing air from passing through the floor, will also greatly assist in resisting the passage of fire. The provisions suggested above will also be of service in deadening the transmission of sound.

Wooden firring and bracketing to cornices, etc. may be avoided by forming them of iron lathed with metal lathing, as shown in Fig. 251.

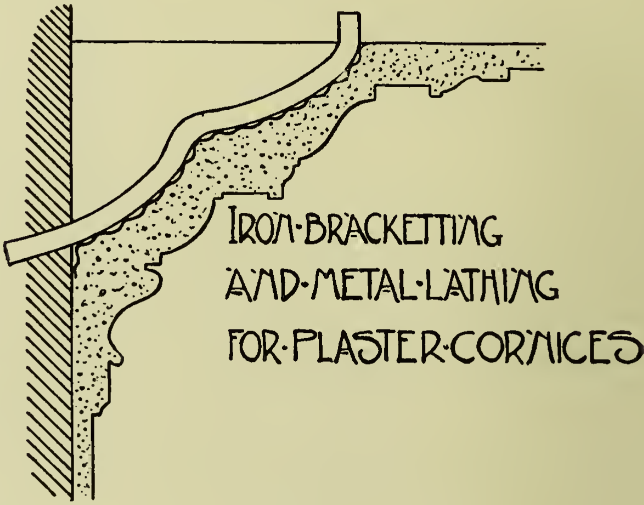


FIG. 251.

The avoidance of air spaces behind skirtings and other linings, by filling in with plaster, is a precaution which can readily be carried out.

The under side of a wooden staircase should receive some form of protection, especially within any cupboard that may be there provided ; for such cupboards are frequently used for the storage of inflammable materials, and are often the cause of fire.

(For the precautions to be taken against Fire in Theatres, see Volume VI.)

END OF VOLUME IV.

Printed by MORRISON & GIBB LIMITED, *Edinburgh*